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ISSUED QUARTERLY

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1947 - 1948

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THE Association was organized in Boston, Mass., on June 23, 1882, with the object of providing its members with means of social intercourse and for the exchange of knowledge pertaining to the construction and management of water works. From an original membership of only TWENTY-SEVEN, its growth has prospered until now it includes the names of 900 men. Its membership is divided into six classes, viz.:

A Member shall be an officer or employee of a public or private water works, an engineer, chemist or other person qualified to aid or interested in the advancement of knowledge relative to water works.

An Honorary Member shall be a person of acknowledged eminence in some branch of water supply or of engineering.

A Junior shall be not less than eighteen years nor more than twenty-five years of age, a student or connected with water supply work.

An Associate shall be either a person, firm or corporation engaged in manufacturing or furnishing materials or supplies for the construction or maintenance of water works.

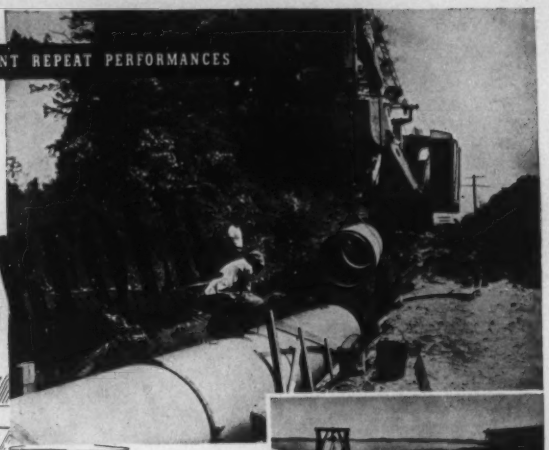
A Corporate Member shall be either a Water Board, Commission, Company or Municipal Corporation.

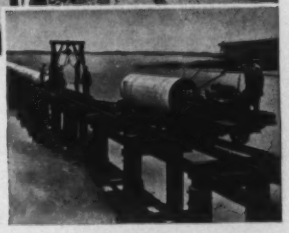
The initiation fees and annual dues are as follows:

	Initiation Fees		*Annual Dues*
Members	$ 3.00	Members	$ 6.00
Juniors	1.00	Juniors	3.00
Associates	10.00	Associates	20.00
Corporate Members	10.00	Corporate Members	10.00

This Association has at least eight regular meetings each year, of which five are held in Boston, one in northern New England, one in southern New England, and one, the annual convention, held in September on such date as the Executive Committee may designate.

LOCK JOINT REPEAT PERFORMANCES





10 JOBS SINCE 1920

AS NORFOLK GROWS so must its water supply system. For more than 25 years municipal engineers have met this City's continually increasing need for water by installing 73 miles of Lock Joint Reinforced Concrete Pressure Pipe. Thus, a water supply system of sustained high carrying capacity, negligible maintenance cost, immense structural strength and unfailing service is assured for generations.

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For future projects, large or small, specify Lock Joint Pipe.

A Hazen-Williams Hydraulic Slide Rule will gladly be sent upon request with our compliments.

1943 – (Top photo) Installing 36" pipe alongside of old wood stave pipeline which it replaces.

1921 – (Lower photo) Installation of Lock Joint pressure pipeline on Trestle over Elizabeth River.

★

SCOPE OF SERVICES

Lock Joint Pipe Company specializes in the manufacture and installation of Reinforced Concrete Pressure Pipe for Water Supply and Distribution mains in a wide range of diameters as well as Concrete Pipe of all types for Sanitary Sewers, Storm Drains, Culverts and Subaqueous lines.

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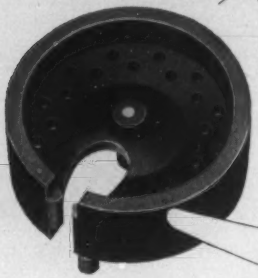

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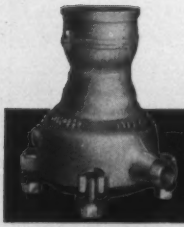
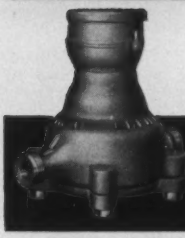


EMPIRE Type 12 METER
Frostproof model for cold climates
Described in bulletin W-802



EMPIRE Type 14 METER
Bronze bottom plate model for warm climates
Described in bulletin W-801

In Empire Type 12 and Type 14 meters the *balanced*, oscillating piston smoothly glides in a circular motion between the halves of a snap-joint measuring chamber. Friction and wear are minimized by having the bearing area distributed over the entire circumference of the piston. And because the web of the piston is centered to equalize pressure above and below it . . . and because of its featherweight, this piston practically floats between the mirror-smooth faces of the chamber.

This exclusive operating principle, unobtainable in any other water meter, has made Empire meters noted for their high degree of sustained accuracy. Even after many years of service, the *balanced*, gliding piston of an Empire will respond to the small flows that an ordinary meter might miss entirely.

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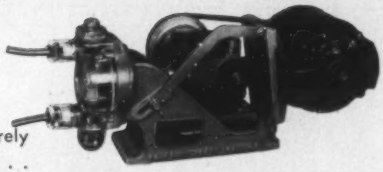

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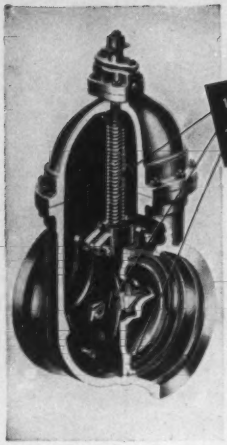
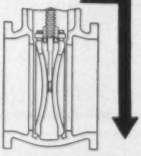


1. Continued turning of valve stem
2. Brings wedges into action and expands the discs against their seats
3. After disc travel ceases.

Why SMITH GATE VALVES

DO NOT WEDGE PREMATURELY

All the Way DOWN



and then OUT

Because of its unique side wedging action, the discs of a Smith Gate Valve are not expanded laterally until after they have completed their travel and fully cover the ports. Termination of the discs' travel and further turning of the valve stem actuates the wedges and expands the discs. In opening, the action is reversed, the wedges are fully released before the discs commence to travel to an open position.

Smith Gate Valves are made in all types and sizes for manual, hydraulic cylinder, or motor operation and for low—medium—high pressure services.

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WATER SERVICE BRASS GOODS

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FLOOR STANDS
INDICATOR POSTS
TAPPING MACHINES
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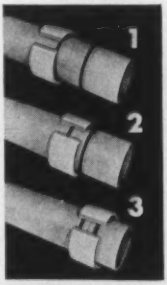
PIPE CUTTING MACHINES

Engineering Facts about
TRANSITE PRESSURE PIPE
Subject:—JOINTS

THE Simplex Coupling—the standard coupling for Transite* Pressure Pipe—was engineered by Johns-Manville to minimize costly underground leakage and to provide maximum protection against pipe line failures resulting from trouble at the joints.

It consists of only three parts: a Transite sleeve, carefully machined to fit over the end of the pipe, and two rubber rings which, when assembled, are tightly compressed between pipe and sleeve. The result is a tight yet flexible seal that provides a number of important advantages.

First, its effectiveness does not depend on the individual skill or training of the workmen. The Simplex Coupling is, in effect, a factory-made joint which is simply *assembled* on the job. Perfect joints are quickly, easily made, even by inexperienced crews. And—a unique feature—each joint may be checked for proper assembly immediately upon completion.

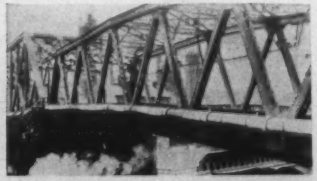


SIMPLICITY of the Simplex Coupling helps assure perfect joints independent of workmen's skill. These cutaway views show complete assembly operation: (1) Start, (2) Sleeve pulled over one ring, and (3) Final position with sleeve centered over joint.

Moreover, the flexibility inherent in the design of this coupling safeguards against leakage once the pipe has been placed in service. Because each joint is capable of deflection, the entire line has a flexibility not commonly found in other types of underground water line construction. This minimizes flexural stresses in the line . . . allows the pipe to conform to soil movement . . . helps it absorb the vibration of heavy traffic under busy city streets. Many thousands of miles of Transite Pipe confirm the ability of the Simplex Coupling to protect the line against the stresses that are so often a cause of joint leakage and pipe failure.

Joints that stay tight in service are but one advantage of Transite—the modern asbestos-cement pipe that was engineered to carry water more efficiently. For further engineering facts, write for brochure TR-11A. Address Johns-Manville, Box 290, New York 16, New York.

DEFLECTIONS up to 5° at each joint are possible with the Simplex Coupling. This permits laying the pipe around curves or obstructions as shown by the installation above.



The flexibility inherent in the Simplex Coupling helps guard against the shock and vibration of traffic. Under busy city streets, this factor also minimizes flexural stresses in the line.

*Transite is a registered Johns-Manville Trade Mark

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... THE MOST ECONOMICAL PIPELINE INSURANCE YOU CAN BUY

Because they possess ability to resist moisture absorption and remain unaffected by the changes in moisture content of soils, Barrett* Coal-tar Enamels provide positive and long-lasting stability of insulation.

These coatings, applied with modern equipment by modern methods, plus electrical inspection, plus cathodic protection, produce corrosion-proof pipelines that require a minimum use of complementary electrical energy and a minimum investment in this form of electrical protection.

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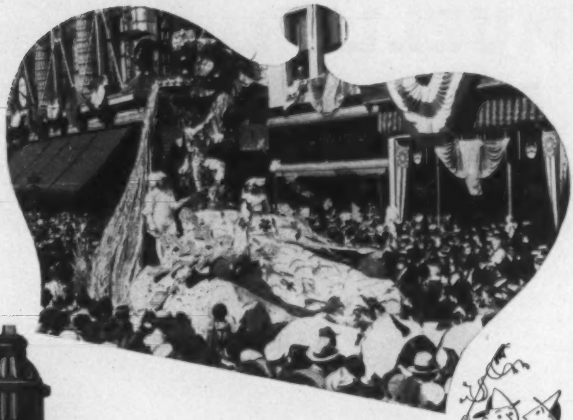
ALLIED CHEMICAL & DYE CORPORATION
40 Rector Street, New York 6, N. Y.

*Reg. U. S. Pat. Off.



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WHAT THESE FELLOWS MEAN is that New Orleans protects her life and property with Mathews Hydrants. That wise old city knows that where there's a Mathews there's extra safety. And why not have it?

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Proved the world around for over 75 years, there's no guesswork about the Mathews. You will want to give *your* community this extra measure of safety. Send for illustrated booklet about the great Mathews Hydrant.

OTHER MATHEWS FEATURES: Head can be rotated 360° • Replaceable head—nozzle outlets easily changed • Nozzle levels raised or lowered—no excavating • Stuffing-box plate integral with nozzle section—strong, safe, leakproof • True compression-type main valve • Only one part to oil—the operating thread.

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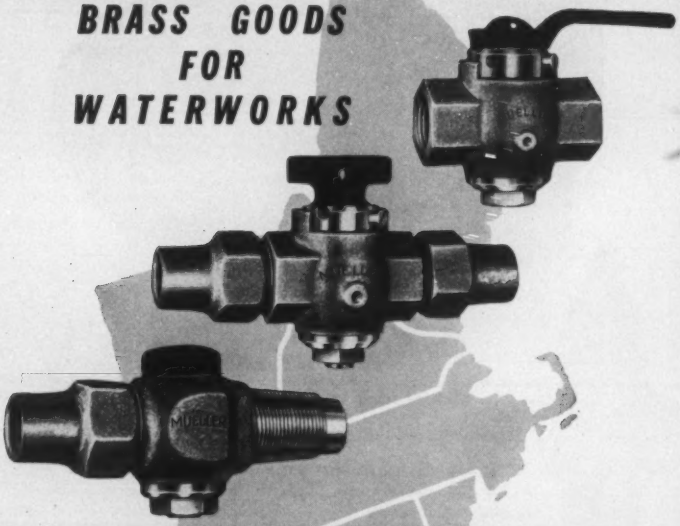
Manufacturers of "Sand-Spun" Pipe (centrifugally cast in sand molds) and R. D. Wood Gate Valves

MUELLER *Blue Top*

BRASS GOODS FOR WATERWORKS



The Mueller BLUE TOP line of water works brass goods is a special line that has been expressly designed and developed to meet unusually hard service conditions. All items in this line are cast with extremely heavy sections from high copper content brass to give maximum resistance to corrosion and to allow for the painstaking grinding and lapping of the key into its body to insure easy turning. This heavy casting also permits of full round way openings and heavy checks—far wider orifices and stronger checks than found on ordinary Curb and Meter Stops without sacrifice to the strength of the body or threads. If you are interested in only the finest waterworks brass goods available, then specify the Mueller BLUE TOP line. Ask any Mueller Representative, or write us direct.

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Badger Meters improve your "customer relations", increase your prestige with taxpayers, assure continued accuracy in registration of all water used. It pays to BUY BADGER.

Every water works superintendent should have a copy of our chart:
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Send for yours today (no charge)

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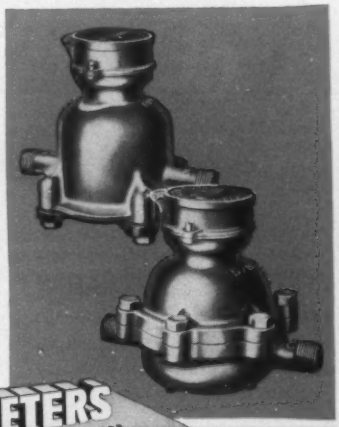

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Only PRELOAD Equipment can Produce PRELOAD Results - - -

Pipe lining surface equipment, Philadelphia, Pa.

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The PRELOAD cement Pipe Lining Machine utilizes material which has been pre-mixed dry, above ground, and is conveyed to it under pneumatic pressure. The mixture is hydrated by water jets at the nozzle immediately prior to being "shot" against the pipe wall with a velocity of over 300 feet per second. The thickness of the cement mortar lining is closely controlled by special mechanism. The cement mortar lining is then smoothed by four graduated spring-loaded trowels attached to pantograph troweling arms.

The surface equipment is designed to operate with a minimum of disturbance to traffic and surrounding business and residential districts; is highly mobile and provides every facility for carrying forward the below-ground pipe lining operation with speed, efficiency, and economy.

PRELOAD Pipe Lining can give a new economic lease of life to old water mains 24" or larger; conserve and forestall capital investments in new pipe lines; improve service by increasing fire fighting, industrial and domestic water pressure; and create new water revenues through increased pipe line capacity.

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A WHAT for every WHERE and WHEN

Preventing and curing water-line headaches is largely a matter of having the right pipe joint or repair product for each specific need. Engineered to solve your problems, Dresser's complete line provides labor-saving, economical products for efficient and permanent pipe joining and repair.

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Where Dresser Products can serve you

Here are some of the many installations on which Dresser Couplings and Repair Products can save you time, trouble and expense:

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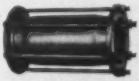
Filtration plants	**Air conditioning piping**
Pump connections	**Sprinkler system piping**

Steel Couplings (Style 38) for joining plain-end steel and cast iron pipe. Sizes from ¾" to 72" and up.

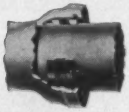
Adjustable Bell-Joint Clamps (Style 60) for repairing and preventing leaks in bell-and-spigot joints. Sizes 3" to 60" CIP.

"Adjustable" Repair Sleeves (Style 82) for quick, permanent repair on cast iron lines. Sizes 4" to 8".

Split Repair Sleeves (Style 57C) for repairing breaks, splits and holes in CIP. Sizes from 2" to 12".



Long Sleeves (Style 40) to span gaps between pipe ends. Simplifies tie-ins. Sizes from ½" to 24".



Bellmaster Joints (Style 85) for Bellmaster cast iron pipe. Fits inside bell. Sizes 3" to 12".

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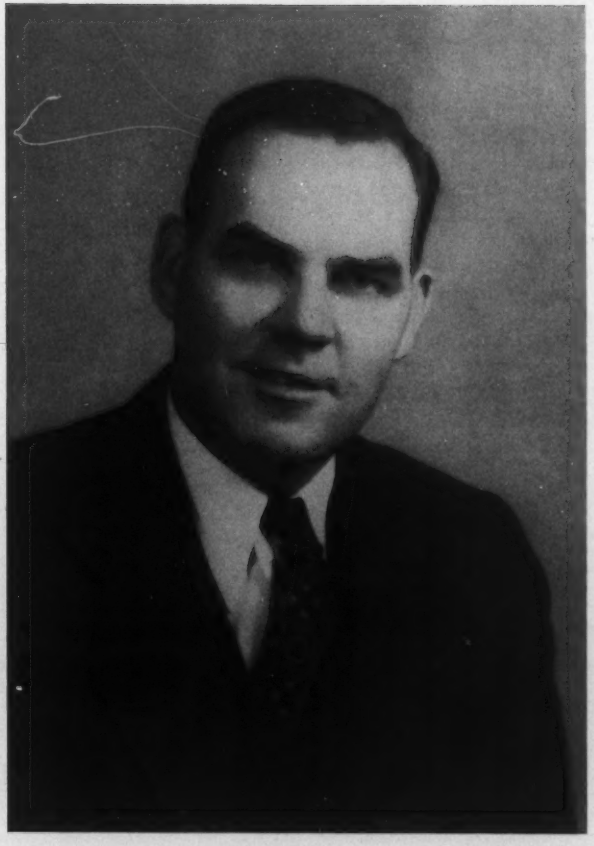

Vogue Studio

DONALD C. CALDERWOOD

PRESIDENT OF THE NEW ENGLAND WATER WORKS ASSOCIATION, 1947-48.

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New England Water Works Association
ORGANIZED 1882

VOL. LXII MARCH, 1948 No. 1

This Association, as a body, is not responsible for the statements or opinions of any of its members

CONTAMINATION OF WATER SUPPLIES BY RADIOACTIVE SUBSTANCES

BY J. CARRELL MORRIS* AND JEANNE B. CARRITT†

[*Read September 16, 1947.*]

THE advent of practical nucleonics through the use of radioactive materials on a large scale will introduce new problems to many industries and professions. Not the least of these are the public health professions, which will be charged with the task of seeing that the use of radioactive materials does not endanger the life or health either of workers in fields employing radioactive materials or of the populace in general. For the sanitary engineer this means, among other things, the development of suitable equipment for the detection of dangerous amounts of radioactivity in water supplies and of suitable methods for the elimination of these dangerous amounts. While it is probable that the problem will be one of immediate interest only in isolated spots for some years to come, nevertheless the critical nature of the problem when it occurs means that every effort should be made to anticipate and solve possible difficulties well in advance of their actual appearance. The present paper makes no attempt to do this in a complete or detailed manner. It is simply an effort to indicate the classes of problems which are likely to be encountered and to suggest possible types of solutions to some of them.

BIOLOGICAL EFFECTS OF RADIOACTIVE RADIATION

Atomic disintegrations involve the release of several types of particles: alpha particles or helium nuclei, beta particles or high-speed electrons, gamma rays or very short wave-length X-rays, neutrons, protons and positrons. Each of these radiations has characteristic effects, but all are toxic to living tissue and produce the unique danger

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involved in handling radioactive materials. The toxic effects may be produced in two ways: (1) by means of general body radiation or direct exposure to harmful concentrations of the radiations, the effects in this case being proportional to the intensity of the radiation and relatively independent of the element which is disintegrating; and (2) by means of absorption of radioactive substances into the body cells or fluids, where they later decompose, giving off the radiations which exert the destructive effects. In this latter case the toxicity of the material depends very greatly upon the extent to which the particular radioactive element is absorbed by the body, as well as upon the intensity of its radiation when it decomposes.

General body radiation produces its toxic results in two principal ways: the gamma rays of the radiation may destroy the body cells—the white blood cells and the blood-forming tissues are most sensitive—or, in those special cases in which neutrons are present in the radiation—for example, the radiation from an atomic energy pile or an atomic bomb burst—the neutrons may undergo radiative capture by the atoms in the body, releasing secondary gamma rays, which then destroy the tissues, and possibly forming radioactive species from the body elements at the same time. Because of their low penetrating power, alpha and beta rays are generally absorbed by the skin and do not exhibit the same degree of toxicity on external exposure as the gamma rays. However, beta rays in particular may cause very bad surface radiation injury. It has been estimated that, if a major portion of the body is exposed, general radiation equal to 400-600 roentgens* is sufficient to cause death, while 50 roentgens or less will cause permanent injury. These figures are for very short periods of exposure. For persons exposed continuously to radioactive materials during working hours, the tolerance limit, or total daily dose which one person can receive, has been set at 0.1 roentgen.*

In considering the effects of the absorption of radioisotopes by the body, the elements can be divided into several categories:

(1) Those elements which do not ordinarily occur as a part of the body and which are not absorbed from the lungs or digestive

*The roentgen is defined as that quantity of gamma radiation which, falling on dry air under standard conditions, produces one electrostatic unit of ion pairs per ml. One may calculate, for example, that about 10^{-10} gram atoms of radioactive sodium 24 would be sufficient to yield the tolerance limit of 0.1 roentgen in 8 hours at a distance of 1 meter.

tract. These elements would be expected to have little effect other than that attributable to general radiation effects. Such elements as aluminum and silicon would fall in this category.

(2) Those elements which occur in the body but only in the fluids, so that they are excreted rapidly. While the materials in this category might be expected to have a somewhat greater toxicity than those in the first group, a highly cumulative effect should not be observed, because of the rapid elimination of the material from the body. Such elements as sodium and chlorine would be expected to fall in this group.

(3) Those elements which occur as a part of the body tissues, particularly those occurring as a part of the bones or marrow. Since these elements may be built into the body structure to be retained at a site where their decomposition will be very destructive, radioelements of this type can be expected to be very toxic and to show a highly cumulative toxicity. This group would include such elements as calcium and phosphorus, which are vital constituents of the bone marrow. The cumulative toxic effect of these elements is affected somewhat by the fact that most of their radioisotopes have relatively short half-lives, so that after a few weeks most of the radioactive material has decomposed. For example, P^{32}, the isotope of the element phosphorus with an atomic weight of 32, has a half-life of 14.3 days.

(4) Elements which, while not normally constituents of the body, are absorbed by it or can replace the usual elements of the body structure. Lead is a good example of this type of element and many of the other heavy elements exhibit similar behavior. Because these elements remain for a long time in the body and often have very long half-lives, their toxic effects are highly cumulative. Plutonium, for example, may be absorbed through both the lungs and, less readily, the digestive tract and deposited in the inner and outer membranes which cover the long bones. Since it is excreted slowly and has a half-life of 24,000 years, it continues to exert a toxic effect as long as it remains in the body. The total tolerance level for plutonium in the body has been estimated to be one microgram.

Means of Contamination of Water Supplies

Contamination of water supplies by radioactive materials under normal peacetime conditions can occur in two ways: through the induced activation, by means of neutron absorption, of cooling water which is run through piles used in the production of atomic energy or of radioisotopes, or through the disposal of waste radioactive materials into water. Other possibilities, which would occur only under wartime conditions, are that the water may be contaminated as the result of atomic bomb bursts or that radioactive materials may be introduced deliberately into water supplies as a form of sabotage.

A. Induced Radioactivity. Up to the present time the greatest problem of induced radioactivity in cooling water has been in connection with the piles for the production of plutonium at Hanford, Wash. In this plant advantage was taken of the fact that hydrogen and oxygen, the elements of water, are both very poor neutron absorbers and form only very short-lived radioisotopes when absorption occurs. The raw water was first treated, to remove from it dissolved or suspended materials containing elements which had a high capacity for neutron absorption or which formed dangerously long-lived radioisotopes. Then the exit cooling water was stored in large concrete retention tanks for a few days, until the radioisotopes of hydrogen and oxygen which had been produced had almost completely disintegrated and the water was sufficiently low in activity to permit its disposal into the Columbia River. However, one point which might be brought out here is that, although this proved to be a satisfactory solution for a single plant, the cumulative effect of the operation of several plants on one water supply might raise the radiation level of the river to a dangerous point.

B. Radioactive Chemical Wastes. With radioactive chemical wastes the disposal methods used so far have been rather similar. At Oak Ridge, for example, the wastes in water solution were discharged from specially designed sinks into large subterranean storage tanks, the required holding period in this case being much longer, because of the greater variety and concentration of the radioactive materials. Disposal of wastes other than those in water has generally

been accomplished by burial deep in the ground or by dumping at sea. Such disposal methods must be resorted to ultimately for all radioactive wastes, for there is no known way of changing significantly the rate of radioactive decompositions.

C. Atomic Bomb Explosions. An atomic bomb explosion yields, in addition to high-intensity neutron radiation and the other radioactive radiations, large quantities of radioisotopes of the elements located in the middle of the periodic table (selenium to lanthanum)—the so-called fission products. Some of these radioisotopes are fairly long-lived and several of them are absorbed by the body, so that they might cause serious contamination of water supplies in the vicinity. In the case of an air burst, such as that at Hiroshima, most of the fission products are carried far upward at the instant of the explosion and are dispersed over a sufficiently wide area so that they are not likely to be a source of major water contamination. This may vary considerably, however, depending upon geography and climatic conditions. For example, an analysis of soil samples in the immediate vicinity of the ground center at Hiroshima showed that the fission products averaged less than 1% of a tolerance dose, whereas at Nagasaki, where there was a moderately strong west wind blowing and a light rain falling, much higher concentrations than this were found about two miles from the ground center. It was suspected that at Nagasaki some persons were affected by constant exposure to the fission products.

Although very little information has been released on the extent and intensity of this type of damage which was caused by the underwater explosion at Bikini, it seems logical to conclude that the effects would be much more intensified than with air bursts. In this case the force of the explosion raises into the air a great volume of water containing the radioactive material, which in an air burst is rendered harmless by wide dispersal at a great height. This very "hot" water, in settling to the earth, would make that land or area on which it dropped uninhabitable for a long time.

A much more likely source of contamination of water supplies from atomic bomb explosions is the production of induced radioactivity in the water by the action of the neutrons released in the explosion. This induced radioactivity would be similar to that pro-

duced in the Hanford cooling water, except that a more persistent activity, resulting from radioactivation of the greater amounts of dissolved or suspended materials in untreated water, would be expected to be produced. Some elements of particular interest in this respect are: iron, which readily absorbs neutrons and has a radioisotope with a half-life of about 47 days; calcium, with an isotope of 180 days' half-life; sulfur, with an isotope of 88 days' half-life, and sodium, with an isotope of 3 years' half-life.

D. Deliberate Introduction of Radioactive Substances. The placing of radioactive substances in a reservoir to "poison" the water supply of a city is a possible method of sabotage. Since this method has never been tested, the following statements are purely speculative.

The choice of contaminant would depend on the following major factors:

(1) The half-life of the element used must be long enough so that it can be processed, be delivered, pass through the reservoir and supply system, enter the body of the consumer and do its damage before appreciable decay has taken place.

(2) Its chemical nature must be such that it will stay in solution under conditions found in the reservoir and the other units of the water works.

(3) The total weight of the material (radio-material and shielding) must be small enough to be conveniently transported. In this connection an alpha emitter would reduce the amount of shielding necessary to protect the saboteur.

(4) Biologically, the radio-substance should be highly toxic and readily absorbable by the body.

Of the readily available alpha emitters—uranium, plutonium and thorium—the most toxic appears to be plutonium. This radioelement has a half-life of 2.4×10^4 years and a tolerance limit in the body of 1 microgram; it is available and its chemistry is known. Present knowledge indicates that the chief mode of entrance of plutonium into the body is through the lungs; approximately 1% is absorbed in the digestive tract; there is probably no absorption through the skin.

In order to provide a lethal dose of 1,000 micrograms* of plutonium for every person in a city of 500,000, 500 grams of plutonium

*Of this amount, 10 micrograms would be absorbed.

would have to be provided. If an individual is assumed to ingest one out of 100 gal of water daily, 50 kilograms of Pu would have to be introduced into the water supply even if there were no storage of the water. It is seen that the amount of Pu that would have to be added to a reservoir to do any damage in a short period of time is very great. It would appear to be simpler to drop a bomb.

Possible Methods of Removal of Radioactive Contamination from Water

It has already been indicated that one practical method for handling water contaminated by radioactive material is to store it until the activity has dropped below the dangerous level. This method, the one being used at Hanford, is applicable only if the activity is due to short-lived radioelements. In order to insure this latter condition with cooling waters being used for industrial piles, the water should first be treated to remove those elements which may form long-lived radio-products. The most feasible method of preliminary treatment would seem to be an ion-exchange treatment, to remove all dissolved ions, together with filtration to remove suspended matter, and deaeration to remove carbon dioxide. To minimize treatment costs, the most efficient scheme might be to recirculate the water after storage had reduced its activity, rather than to discharge it to waste. This would also avoid any contamination of the water supply.

The ion-exchange technique might also be used to remove radiocontamination from the waste water or from chemical wastes, since zeolites and ion-exchange resins will function for concentrations as low as 10^{-12} molar. This method should work with any ions but would fail with radioactive hydrogen and oxygen of the water itself or with dissolved gases, such as carbon dioxide, containing radioactive atoms. From a purely speculative viewpoint, the ion-exchange procedure would also seem to be the best available technique for the removal of fission products from contaminated waters.

If the radioactivity of the water is known to be caused by certain specific elements, then the activity can be largely removed by the so-called "carrier" technique. This consists of adding to the water a considerable amount of the naturally occurring non-radioactive form

of the element or elements causing the activity and then precipitating out the radioactive isotope along with the added element. For example, if radioactive ferric iron were present, one would add several ppm of ordinary ferric iron and then precipitate the whole as ferric hydroxide. The per cent reduction by this method is determined by the ratio of the solubility of the precipitated ion to the amount of non-radioactive ion added.

Finally, it may be possible in certain circumstances to remove a large percentage of the radioactive substances by modifications of the normal coagulation process. Since many ions are rather highly adsorbed on, or co-precipitated with, aluminum hydroxide, the formation of a floc of $Al(OH)_3$ and its removal by filtration might carry down at the same time considerable amounts of dissolved radioactive elements. This would probably be a rather selective effect, however, and considerable experimentation would be needed to establish its value for various radioactive substances.

With any of the last three possibilities, one is left with a highly radioactive material which must be disposed of. In the first of these it is a highly active zeolite or resin bed; in the other two it is a highly active sludge. The zeolite or resin bed could be regenerated by normal methods, the activity being transferred to the regenerating solution, which would then have to be buried or dispersed in some fashion. The same burial or dispersing procedure would probably have to be used with the active sludges.

While all of these treatment methods except the first one are in the realm of pure speculation, they do at least indicate that the treatment of radioactive contamination of water supplies is not an insuperable one for the sanitary engineer. It can be seen that, if the same degree of ingenuity is applied to this problem as has served to solve so many sanitary questions in the past, then usable and efficient solutions to it certainly can be found.

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Discussion

Arthur L. Shaw (Boston, Mass.). I thank you, Dr. Morris, for a very fascinating, factual and speculative paper.

William W. Brush (New York, N. Y.). Is there any reason to assume that our present water supplies have radioactive material in them that requires consideration?

J. Carrell Morris. All waters contain very, very slight amounts of radioactivity but the amounts are extremely small. Even in the so-called radioactive springs, which were sometimes bottled as beneficial mineral waters, the radioactivity was so slight that it had absolutely no effect.

ECONOMICS OF REHABILITATION OF 3-INCH TO 24-INCH WATER MAINS

BY ALFRED B. ANDERSON*

[*Read September 17, 1947.*]

THE loss of carrying capacity in water mains has been the subject of study for a number of years. Numerous articles have been written on the subject, pointing out the advisability of knowing the condition of one's water system. Other publications have covered thoroughly the increased cost of pumping water through a tuberculated system, showing the increase in cost as related to the decrease in the coefficient.

Verification can be found in professional reports to substantiate a loss of 64-66% of the carrying capacity of the more common sizes of pipe in a period of 30 years, due mainly to tuberculation.

There is an ever-increasing number of pipe lines rapidly approaching a condition of serious inadequacy, because of a lack of hydraulic capacity after a number of years of normal operation. Certain types of water treatment are contributing factors in this accelerated rate of capacity loss.

When one considers that two-thirds of the cost of a water system is in the distribution of the water and that 80% of this value is represented in the mains and valves, is it not reasonable to assume that this particular section of the system is entitled to a reasonable expenditure for maintenance? Most water utilities have funds set aside each year for the maintenance of buildings, filters, pumps, regulating valves, etc., but when it comes to the distribution system, the money set aside is for expansion and replacement.

In the design and financing of pipe-lines the accepted practice has been to consider as a base the thirty-year period. Early data indicated a capacity loss of 30-35% as an average for this period. More recent data tend to raise this percentage to 50-80% in small lines in many instances.

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This obsolescence in the distribution system is seldom recognized under ordinary conditions. It comes as quite a shock to the water superintendent, when the Board of Fire Underwriters conducts flow tests on the system and finds it decidedly under capacity, or—an even more serious matter—when a fire occurs in a congested neighborhood, resulting in the loss of several buildings.

Such a fire was recently experienced by a community not too far from New York City. When the system was installed about 1903, ample pressure and volume were provided to take care of all domestic needs as well as the requirements for fire fighting. At that time, according to the narrator of the story, there was sufficient pressure to tear the shingles off the buildings. Some years later a fire pumper was purchased, because sufficient pressure from the fire hydrants was lacking. During this past year, four buildings were burned to the ground, due to a lack of water to fight the fire, despite the fact that the standpipe was filled at the time. Needless to say, a pipe-line rehabilitation program is under way for this community. When this program is completed, it can be conservatively stated that the capacity of this system will be better than 95% of its original capacity and the cost to the community will be about one-tenth of the cost of replacement.

Inasmuch as recent data indicate a loss in carrying capacity for small mains of 50-85%, let us assume an average of 60%. In this case, the loss in carrying capacity per year is approximately 1.5%. The cost of rehabilitation for the same period would be approximately ¼% per year.

If a fixed amount of money could be set aside each year for the rehabilitation of the distribution system, it could be carried as an asset for the company or department. This may be considered as prepaid insurance against loss of carrying capacity, making funds available when rehabilitation is required. Furthermore, since the life-expectancy of cast-iron pipe is constantly increasing, it seems only reasonable to state that the value of the pipe in the system will be restored—if not fully, then to nearly the original value—and this restoration in turn increases the assets of the water company.

One way of approaching the problem of pipe-line rehabilitation for sizes from 3 in. to 24 in. is the Pittsburgh-Eric process. This

method was originated, developed and perfected by the Eric Engineering Co. of England some fifteen years ago and has been commercially successful for the past ten years. During this time 2,500,000 ft of water mains have been rehabilitated by this company.

About a year ago the Pittsburgh Pipe Cleaner Co. obtained the patent rights for this method of processing water mains in this country and Canada. The reception and interest shown by the water-works profession have been most gratifying and a number of reconditioning programs are under way.

The economy of this process can best be explained by the following facts affecting the cost:

1. Maximum lengths of pipe that can be coated are as long as 1,000 ft and, under some conditions, this length may be extended.

2. A line is out of service for short periods only.

3. Corporation taps, cross connections and other fittings are not dug up nor exposed. The LECTUMEN is flushed out after coating.

4. Pipe is immediately available for use after coating process is completed. A few minutes' flushing is required before restoring pipe to service.

5. The method is virtually fool-proof. No possibility exists whereby a main can be blocked. Thus, mains can be rehabilitated in congested areas with confidence.

6. In reconditioning transmission mains which must be kept in operation, temporary lines are laid.

7. The electrolyte is an emulsion of bitumen and does not require a dry pipe.

8. The process deposits a smooth, even coating of bitumen over the entire pipe interior, which will retain its high carrying capacity for many years.

9. Coating does not materially reduce the cross-sectional area of the pipe.

10. Coating imparts no taste or odor to the water and is non-injurious.

Briefly described, the Pittsburgh-Eric process is an electrolytic method for depositing an especially prepared bitumen compound in aqueous solution on the pipe interior. In principle, the process is

similar to well-known copper- or silver-plating electrolytic methods, with minor exceptions.

In the electrical hook-up, cables from the generating unit are connected to the pipe—anode—and to the cathode, which is drawn through the main. By inserting the cathode in the electrolyte, contained in the pipe, and completing the electrical circuit, the coating begins at once. Molecules of bitumen are uniformly deposited on all metallic surfaces within the area set up between the cathode and anode. Coating is completed when a sufficient thickness of bitumen is deposited to prevent the passage of current within the field.

While this explanation illustrates the principle involved when the cathode is stationary, the continuous coating operation varies somewhat in actual practice. The same principles are involved and the process is instantly controlled from the instrument board. The ammeter and voltage readings constantly reveal to the operator the exact condition of the processing.

The forward movement of the cathode is predetermined and its speed is based on the size of the pipe plus other calculable factors. Any variables are communicated to the operator by instrument board and adjustments are made to compensate for changes. The speed of travel of the cathode depends on operating conditions and ranges from 2 to 6 ft per min, depending on the size of pipe.

As previously stated, the coating is positive. All metallic areas and even lightly coated sections of old non-mineral coatings in the pipe interior are completely covered with bitumen. Areas which are not completely clean will not hold a coating, even if deposition occurs, if such sections contain mineral deposits.

It is imperative, therefore, that the pipe be clean. A completely new type of tool is used. This unit employs a sectional cleaning design, whereby the removal of deposits is positive and definite. Plows, scrapers and brushes under high pressure break away the deposit, dig out the pits and brush the entire interior in one operation. The design of equipment permits "on-the-job" adjustments to compensate for changing conditions.

The engineering problems involved in reconditioning work are relatively simple. After the extent of the work is determined, the openings are indicated on drawings, after due consideration of supply

problems. These locations are later marked in the field and a schedule of work is developed. From this information, the consumer can be notified as to when he will be without water and the fire department likewise advised of the date and location of work. The operation of supply valves and exposing the pipe are normally part of the engineering considerations.

The equipment required in the cleaning and processing operation consists of electrical generating equipment, pipe cleaners, pumps, cutting equipment, tanks and incidental accessories.

A section of the main is opened at each end by removing a short length of pipe. After cleaning, the cathode is pulled to the extreme end of the line, while the pipe itself is joined to the electrical circuit forming the anode. Each end of the section is capped with a fitting, which holds a short length of pipe that extends at an angle to the street level. The electrolyte is pumped into the line until it is completely filled. With this preliminary work completed, the current is turned on and the process begins.

In addition to the electric supply cable, a wire-pulling rope is attached to the cathode. These terminate in separate winches, which are located on the chassis of the generating unit. The cables are reeled on the drums of the winch, either automatically or manually, at a given rate of speed. Under normal operation, coating is continuous until completion, or the moment the cathode has traversed the entire length of the pipe. The electrolyte is then pumped from the system and the section is flushed with water for a few minutes, before being placed in service.

The coating is tough—approximately the thickness of a good tar-dipped coating—and will not crack. It adheres tenaciously to the pipe wall, where it has been evenly and smoothly deposited—even to the ends of the joints.

In the light of cold facts, reconditioning by the Pittsburgh-Eric process is sound. Any size of main in the range mentioned can be cleaned and restored to full carrying capacity. Coating protects the interior surface, increasing the useful span of life. From an operating viewpoint, money available for replacement can largely be diverted to expansion or other improvements.

Discussion

Percy A. Shaw (Manchester, N. H.). Is it necessary to pump some water into each of the services, to clean out the ends of the service pipes?

Alfred B. Anderson. No, sir. As far as the service connections are concerned, the general practice is to close the valve at the curb, the curb cock, if you will. As your LECTUMEN is put into the pipe, there is a certain amount of air entrapped in the service itself.

Richard H. Ellis (Boston, Mass.). I think this is a very interesting process which he has described. I think it might add appreciably to the discussion if Mr. Anderson could give us some information as to the length of time in which this coating might be expected to be effective. That is based, of course, on the English experience. We have to appreciate that the experience in this country has been very limited to date.

Mr. Anderson. We know that the coating will stand up for ten years and that is as old as the operation. We had a small piece taken at random out of a system, that had been coated ten years previously. There were two little tuberculations, probably the size of your small fingernail, on 18 in. of pipe after ten years.

Mr. Ellis. I personally saw that piece of pipe and was rather impressed.

Mr. Anderson. After that ten-year period, your guess is as good as ours. We felt that the coating was in fair condition at that time and should be good for another ten years at least.

Mr. Ellis. Have any tests been made to determine the coefficient in existing lines, that you know of?

Mr. Anderson. I don't know of tests in England but we had a test here the other day from the Philadelphia Suburban Water Co. We did a job down there for them on a 6-inch pipe whose coefficient was, before cleaning, 49, and after it was completed the coefficient was 128.

Mr. Ellis. My point was more a comparative test over a period of 8 or 10 years.

Mr. Anderson. We have no figures on that, I am sorry to say. We hope to get them.

E. SHERMAN CHASE (Boston, Mass.). What would the approximate cost run per linear foot of the various sizes?

MR. ANDERSON. The 4-, 6- and 8-inch pipe will be under a dollar. We do not like to quote actual prices, until we know just exactly what we are going to be required to do but, for the 4-, 6- and 8-inch we generally come under a dollar. The 4-inch pipe, including the municipality's cost, would not run much over 75 or 80 cents. That is the total cost, municipal and the contract. Of course that will vary, too, if you have paved streets as compared to dirt streets. There is quite a difference of cost with the pavement.

ARTHUR L. SHAW (Boston, Mass.). Does the deposit of bitumen on the bronze seats of gates and on bronze gate stems and threads cause any difficulty?

MR. ANDERSON. We cannot coat the threads nor the bronze on those, and hence we have not been able to determine that there was any difficulty. It seems that there is a polished surface and the polished surface does not take a coating.

A MEMBER. Does the iron have to be thoroughly cleaned before the coating is applied?

MR. ANDERSON. Yes, it definitely must be clean. There has been a slight change in the thought regarding cleaning, since we have been in this business. Previously, the idea of cleaning the main was to take the tubercles out of the pipe and leave the old coating in place as much as possible. I have even heard of companies which have attempted to put a rubber bearing-surface on scrapers, so that the old coating would not be disturbed. In this case, you have to take the tubercles out—you have to dig out the pits—and in so doing we do take off practically all the old coating.

We had an interesting job in Baltimore not too long ago on a 12-inch salt-water line, which contained probably 1½-2 in. of tuberculation. We had to clean that line seven times before we got down to the original metal. The coating will not stick on pipe that has been through the process of graphitization.

WILLIAM W. BRUSH (New York, N. Y.). In regard to the graphitization of the pipe and the adherence of the bitumen in the Eric process to a graphitized surface—the pipe which was brought over from England, which I personally examined, showed the entire

surface was graphitized before the Eric process was applied. That pipe had been used for ten years and apparently showed, as Mr. Anderson and Mr. Ellis said, no attack during that ten-year period. But there was definitely graphitization of the entire surface of the interior of the pipe.

I ask the question, therefore, in regard to this salt-water pipe, if the graphitized iron was removed down to the original iron that had not been graphitized.

MR. ANDERSON. The answer to that, Mr. Brush, is that this salt water had gone so far that the surface of the pipe was just like so much putty. You could scrape it with your fingernail. We had to get down to the hard metal. We had to dig off that ¼-⅜ in. of soft putty material. We did not actually get to the bottom of the effect of the graphitization. There is also a layer on the inside of graphitization that showed on the English pipe. We could go down approximately to the top of the metal shown on the English pipe and there was still a little discoloration under the graphitization, after we had taken it off.

A MEMBER. Was that bitumen coal-tar or asphalt?

MR. ANDERSON. Asphalt.

RELEASES FROM QUABBIN RESERVOIR USED TO DEVELOP POWER

BY CHESTER J. GINDER*

[*Read September 18, 1947.*]

THE construction and operation of a power plant, named the Winsor Station, at the Quabbin Reservoir of the Boston Metropolitan Water District is of interest, I believe, inasmuch as the reservoir was constructed for water-supply purposes and the power is to be developed from releases to the Swift River, which are required by the War Department and Massachusetts legislation.

The District is authorized by Massachusetts legislation to take from the Swift River (on an area of 186.0 sq mi) the entire flow, except that a sufficient quantity must be released from the reservoir to insure the maintenance of a flow of 20 mgd into the mill-pond at Bondsville, located about 5 miles downstream from Winsor Dam. This means that, regardless of what the natural flow of the river may be, there must be discharged each day from the reservoir at Winsor Dam a flow of 20 mgd less the yield of the small intermediate drainage area of about 8 sq mi above Bondsville and below Winsor Dam.

The District is authorized by Massachusetts legislation to take from the Ware River at Coldbrook only the flood flows in excess of 85 mgd—that is, on any day when the natural flow from the 96.8 sq mi does not exceed this rate no diversion can be made. Furthermore, this diversion cannot be made in the dry months between June 1 and December 1, except with the permission of the State Department of Public Health.

In addition to these restrictions placed upon diversion by Massachusetts legislation, the War Department, in the interest of navigation on the Connecticut River, does not permit any diversion to be made from the Ware River between June 15 and October 15 and requires under certain conditions of drought a release of additional water from Quabbin Reservoir.

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The present water-supply demand of the District is less than the safe yield of the Quabbin Reservoir and for some years the releases to the Swift River will be in excess of the 20-mgd minimum. As these releases are to be used for the generation of power, the station will be amply supplied for a considerable time to come.

The development of the Quabbin Reservoir as an extension to the water supply of the Boston Metropolitan District has been described in detail in numerous articles and papers and only a brief description and reference to its location will be given here.

The Quabbin Reservoir, the latest extension to the District's supply, is located in the valley of the Swift River, a tributary of the Connecticut River, 66 miles west of the State House, and has a natural, tributary drainage area of 186.0 sq mi. The reservoir, created by two large earth dams, is 18 miles long and has an area of 38.6 sq mi and a storage volume of 415 billion gallons. The construction of the reservoir as part of the extensions recently built by the Metropolitan District Water Supply Commission—now the Construction Division of the Metropolitan District Commission—was authorized by Chapter 375 of the Acts of 1926 and by Chapter 321 of the Acts of 1927. The project also included the construction of a tunnel about 24 miles long from Quabbin Reservoir to Wachusett Reservoir and the construction of a diversion dam and an intake structure on the Ware River, where it passes over the tunnel at about its midpoint. On the Ware River, which is also a tributary of the Connecticut River, construction included provision for passing the flood flows of the Ware River through a shaft about 260 ft deep into the tunnel and thence into either the Quabbin Reservoir to the west or the Wachusett Reservoir to the east, as operation of the system requires. The upper portion of the Ware River, 96.8 sq mi, is therefore made tributary to Quabbin Reservoir by this connection.

The safe yield of the natural drainage area, together with the area made tributary to it by diversion, amounts to 195 mgd and the 415-billion-gallon capacity of Quabbin Reservoir is very large in proportion to these areas. The extra large storage capacity was provided, first, to continue the policy of the District in relying principally on long-term storage for quality and, secondly, to anticipate diversions from additional watersheds in the future, although

there is no present authorization for any such additional development.

During 1946 the 20 cities and towns which are members of the Metropolitan Water District, of which 18 take their entire supply from District sources, consumed an average of about 178 mgd. Quabbin Reservoir, therefore, could supply the present needs of the District without assistance from the other sources of the system.

Power Plant

The power plant is located in an outlet structure, about 800 ft downstream from the center line of the Winsor Dam, on the westerly side of the Swift River. The releases from the reservoir reach the outlet structure through two conduits, about 900 ft in length, built into a by-pass tunnel, which was used for river diversion during construction of the dam. One is a 48-in. steel-plate pipe with mortar lining, which was built in an enlargement in the by-pass tunnel section and entirely encased in the tunnel lining. The other is a 68-in. steel-plate, motar-lined pipe, enclosed in a concrete envelope and placed in the by-pass tunnel, after the need for by-passing the river around the dam had ceased and after the tunnel had been plugged at the upstream end. The tunnel, which is about 1,300 ft long, pierces the rock which forms the west abutment of the dam. Near the point where the by-pass tunnel passes under the center line of the dam, a shaft was built to the surface and in the wells of this shaft are located valves, operated in the intake-works structure above, which will permit the control of the flows to be released from the reservoir through both the 48-in. and 68-in. lines.

The outlet structure which houses the power plant, shown in Figure 1, is located on the downstream end of the by-pass tunnel and valves on the 48-in. and 68-in. lines in the outlet structure also permit the control of releases from the reservoir at this point. The 48-in. and 68-in. pipe-lines through the by-pass tunnel therefore constitute the penstock for the power plant. The 48-in. line was used to release water down the river during the time the reservoir was filling, whereas the 68-in. line, which will serve communities downstream from the dam with domestic water, if and when proposed plans are carried into effect, has not been used to any extent.

Water-supply purposes required cross-connections between the

FIG. 1.—THE OUTLET STRUCTURE WHICH HOUSES THE POWER PLANT.

68-in. conduit and the 48-in. conduit, so that the outlet structure contains a rather complicated arrangement of large piping, hardly suited to best advantage for power-plant purposes and with only a minimum of space available for the waterwheel and generator. Inasmuch as the development of power is incidental to the water-supply requirements, the losses in the intake and in the penstock piping have been given only secondary consideration.

The flow line of the reservoir is at elevation 530 with respect to the zero of Boston City Base and the center-line of the turbine scroll-case is at elevation 390.17, a distance of 140 ft below the crest of the reservoir spillway.

The water in Quabbin Reservoir was first impounded in 1936 and releases down the Swift River through the 48-in. pipe-line began at that time. The foundations for the generator and waterwheel had been partly built under an earlier contract and the releases were by-passed under the location for the waterwheel and into two precast-concrete discharge conduits to the channel, downstream from the outlet structure, and thence into the Swift River. Each of these conduits has Venturi-meter castings built into it for the measurement of the releases, the larger Venturi being 60 in. by 33 in. and the smaller 33 in. by 19 in. The 60-in. conduit now constitutes the tail-race for the turbine. The draft tube from the turbine discharges directly into this 60-in. conduit and 96 ft farther downstream the flow passes through the larger Venturi meter and then, after a short horizontal run, discharges vertically through a 90° bend in the bottom of the outlet channel.

Immediately downstream from the point where the draft tube enters the 60-in. pipe-line, which forms the tail-race, there is an access chamber, built over a 24-in. manhole into the 60-in. pipe. This served for inspection purposes in connection with the 60-in. conduit. Since the installation of the power plant, the bolted cover on the 24-in. opening has been removed and, as the hydraulic gradient is such that the free water surface stands up in the access chamber when the turbine is running, the chamber will act as a surge tank in the event of sudden changes in the turbine gate-openings.

At the time the foundations for the Outlet Works Building were built, the lower portion of the draft tube was constructed and

capped with a steel-plate cover. When the superstructure was built, the floor area to be occupied by the generator was built with removable checkered plates. These were taken out when the power plant was installed; the draft tube was extended and the foundation for the waterwheel and generator was constructed up to the level of the operating floor. Some revisions and changes in the piping in the basement were necessary, to accommodate the waterwheel and appurtenances as furnished by the manufacturer.

Hydroelectric Service

The Winsor Station will be the third hydroelectric power plant to be operated by the Metropolitan District Commission, the first having been built at Wachusett Dam in Clinton and the second at Sudbury Dam in Southborough. A fourth is in process of construction at Shaft 1 of the Quabbin Aqueduct, where the flow of the aqueduct enters Wachusett Reservoir.

The generation and sale of power as a by-product in connection with the operation of the Metropolitan Water Works was provided for by legislation in 1895. The Wachusett hydroelectric power station was constructed in 1911 and is believed to be the first plant where public water supply was utilized in this manner. In 1916 a power plant was placed in operation at the dam at Sudbury Reservoir. These two power plants are operated on water required for water-supply purposes and the same will apply to the future power station to be installed at Shaft 1 of Quabbin Aqueduct, whereas the Winsor Station on the Swift River operates on waste water from the reservoir, discharged to the river below the dam.

General Description of Equipment

The waterwheel-equipment installation consists of a single-runner, vertical hydraulic turbine, manufactured by the S. Morgan Smith Co. and arranged for direct connection to an electric generator. The single runner discharges downward into a draft tube and then into a 60-in. conduit about 100 ft long, which constitutes the tail-race between the power plant and the discharge channel. The gates are of the balanced wicket type, operated by an oil-pressure governor, working through a torsional gate shaft and linkage, located on the

generator floor. The weight of the rotating parts of the turbine and the hydraulic thrust on the top of the runner are carried by a thrust bearing on the top of the generator. The manufacturer has guaranteed that, when operating at a speed of 600 rpm, the wheel will supply not less than the following power under the net effective heads designed:

Head	*Horsepower*
145 ft	1,680
125 ft	1,320
100 ft	915

It is expected that efficiencies such as the following will be obtained:

145-ft head			125-ft head			100-ft head		
Load	Hp	Eff. %	Load	Hp	Eff. %	Load	Hp	Eff. %
100%	1,860	84.9	100%	1,460	84.8	100%	1,015	83.8
Best	1,430	87.8	Best	1,210	89.7	Best	930	88.4
75%	1,395	87.7	75%	1,095	88.5	75%	762	87.0
50%	930	81.8	50%	730	82.2	50%	508	80.9

The turbine shaft has one bearing of the babbited type, mounted on a crown plate and arranged for oil circulation.

The runner, made of cast steel, is 26 in. in diameter and was cast in one piece. The casing, which is of spiral form, is made of cast iron in one piece, without circumferential joints, and has an entrance diameter of 36 in.

The turbine is equipped with a 6,700-foot-pound Woodward governor, located on the generator floor and arranged to operate the gates by means of a torsional gate shaft. The governor is supplied with oil at a pressure of 200 lb per sq in.

The governor consists of a centrifugal speed element, driven by a motor, and is complete with hand control, load-timing device, gate indicator, synchronizing attachment and all necessary anti-racing mechanism between the gate-operating mechanism and the governor.

Pressure Regulator

The turbine is also equipped with an 18-in. pressure regulator, which is mechanically connected to the gate mechanism of the turbine and operated in synchronism with the gates. It closes automatically after a load change and the maximum pressure due to full load change on the turbine will not exceed 25% above normal. This value is based on a two-second governor stroke. The regulator is so designed that it is impossible for it to close accidentally nor can any derangement of the control mechanism cause it to close more rapidly than the normal rate of operation.

The runaway speed at no load and 145-ft head is 1,220 rpm. The waterwheel is designed for left-hand rotation and the weights given by the manufacturer are 10,000 lb for the scroll case, which is the heaviest piece, and 40,000 lb for the total weight.

Generator

The generator, shown in Figure 2, was manufactured by the Electric Machinery Manufacturing Co., Minneapolis, Minn., and is the vertical type, direct-connected to the hydraulic turbine and also direct-connected to the exciter and permanent-magnet generator for driving the governor flyballs. It has a rating of 1,500 kva, 0.80 power factor, 2,400 volts, 3-phase, 60 cycles at 600 rpm.

The permanent-magnet generator is a relatively new device for furnishing power for driving the governor flyballs. As it is direct-connected to the turbine and generator shaft, and electrically connected to the governor-head motor, the latter will start rotating as soon as the prime mover starts and in exact synchronism with it. Thus the unit is always under governor control.

Other Equipment

The waterwheel and appurtenances, together with the generator with its protective devices, constitute the more important parts of the equipment items at the station. However, the completed installation included rather complicated switchgear to handle the energy generated, together with an outdoor transformer substation at the rear of the outlet-works structure, a remote-control panel at the nearby

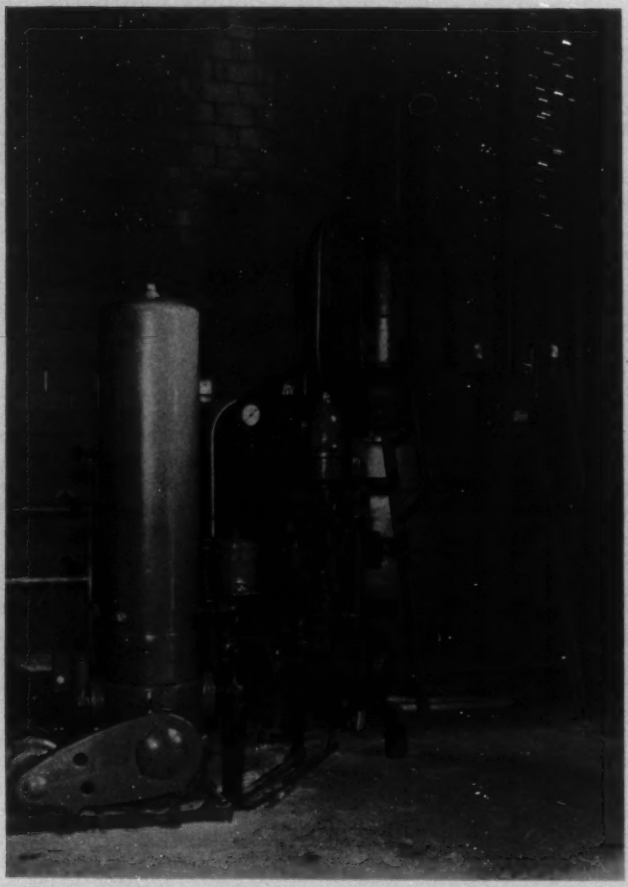

FIG. 2.—GENERATOR AND GOVERNOR MECHANISM.

Administration Building and a 22,000-volt pole transmission line, about 570 ft long, running from the substation to the connection with the lines of the New England Power Co. The transmission line was furnished and erected by the installation contractor but all of the principal items, including the turbine, generator, switchgear, transformer and substation structure, were purchased by the Commission and installed under a separate contract.

The switchgear, shown in Figure 3, was manufactured by the General Electric Co., Philadelphia Works, and is the metal-clad type. It is in the power-plant structure and provides for fully automatic or manual control of the generator and the feeder circuit to the substation. Further control is provided by means of a panel located in the Administration Building, at which point complete remote operation of the unit is possible. This remote-control panel is equipped with a master switch for starting and stopping the unit, a governor motor control switch for varying the load, push-button stations for opening and closing two motor-operated valves on the penstock lines, and an alarm which operates in the event of a shutdown by reason of some failure, as well as indicating lamps which show the position of the circuit-breakers on the switchgear in the power plant. Many protective features have been incorporated into the equipment controls, which will prevent starting the unit or shut it down temporarily, in case of either mechanical or electrical trouble in any part of the power plant.

Substation

The substation, shown in Figure 4, includes an outdoor-type transformer, having a rating of 1,500 kva, 3-phase, 60 cycles, 2,400 to 22,000 volts, with tap changers for varying the voltage output. It is oil-insulated and self-cooled, with conservator-type tank. The Erie Electric Co., Inc., of Buffalo, N. Y., was the manufacturer.

Metering is done on the high-voltage side, for which a General Electric Co. outdoor metering outfit is provided. One watt-hour meter registers power sent out to the New England Power Co. and another the power obtained from the New England Power Co., when the station is shut down.

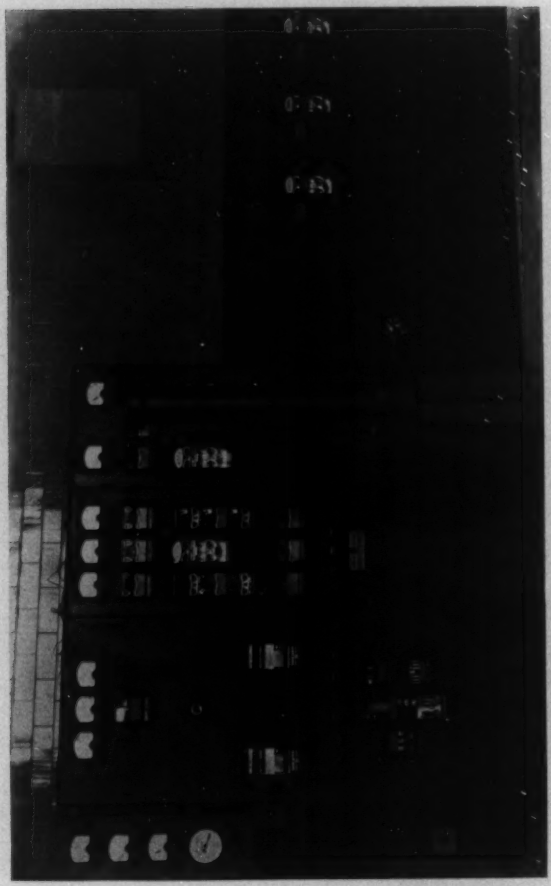

FIG. 3.—METAL-CLAD SWITCHGEAR IN POWER PLANT.

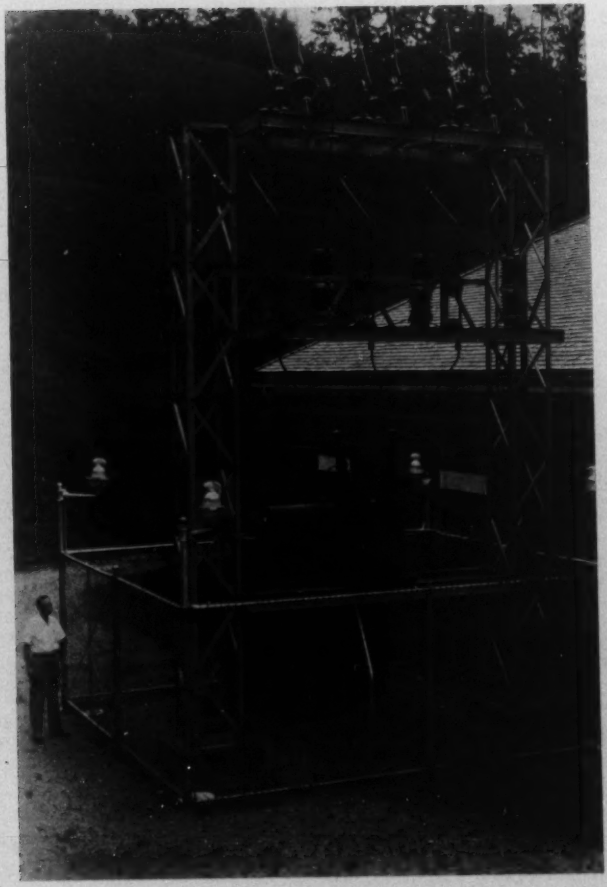

FIG. 4.—OUTDOOR SUBSTATION.

Cost

The work of installing the power plant has progressed to the stage where test runs are being conducted and, although all costs for the job have not been determined, the cost will be approximately as follows:

Equipment		
Hydraulic turbine		$27,953.00
Generator		17,192.00
Switchgear	$10,403.00	
Bus reactor	450.00	
Metering outfit	803.00	
Control panel	554.00	12,210.00
Substation structures, switches and miscellaneous supports, insulators and fittings		1,833.00
Storage battery and charging equipment		965.00
Transformer		4,299.00
Total Cost of Equipment		$64,452.00
Installation		
Waterwheel, governor, etc.		$4,900.00
Generator, exciter, etc.		3,100.00
Switchgear, controls, etc.		1,500.00
Erecting outdoor substation		2,000.00
Furnishing and erecting 22,000-volt transmission line		3,000.00
Furnishing and installing conduits, wiring, etc.		15,750.00
Cement, steel, concrete, etc.		8,948.10
Total Installation Cost		$39,198.10

It is estimated that, when engineering and contingency costs are fully determined, the total cost for the power plant will amount to about $115,000.

Sale and Purchase of Power

The Commission executed a contract with the New England Power Association for the sale and purchase of power at the Winsor Station. This contract can be cancelled on six months' written notice by either party and it provides for the purchase by the power

company of so-called primary power during "Specified Periods" at $0.00625 per kilowatt-hour. Other provisions of the contract include the following requirements:

"The Commission shall so arrange the discharge of water at the station that so much as possible of the electricity shall be available during Specified Periods, as hereinafter defined, and shall, in so far as possible, deliver such electricity to the company during Specified Periods up to the full capacity of the station.

"Specified Periods shall be such periods as the company may specify from time to time but shall in any event include not less than seven hours of each Saturday.

"The Commission shall sell and the company shall buy all the electricity generated at the Winsor Station with the exception of that used in connection with the operation of the station, Winsor Dam, the Quabbin Hill Buildings and the Administration Building; provided, however, that the Company shall only be obliged to take electricity during week-days, excluding Saturdays and legal holidays, and during a period of sixteen consecutive hours commencing at 7:00 A.M. of each day and also during such Specified Period as shall not fall within such sixteen hours.

"Although not an obligation it is the intent that the electricity generated at the Winsor Station during hours other than those specified above will be purchased by the Company if it can be economically absorbed in the Company's system at the price at which it is offered by the Commission from time to time.

"The Commission shall operate the Winsor Station in parallel with the Company's system at such power factor, not less than 80%, and at such voltage as the Company may reasonably desire.

"Whenever the Winsor Station is not in operation the Commission may take from the Company such electricity as may be required in connection with the operation of said station and the other adjacent water works buildings, it being understood that such requirements will aggregate not over 100 kilowatts.

"All electricity shall be delivered in the form of three-phase sixty-cycle alternating current at a pressure of approximately 22,000 volts at the point on the Ware-Belchertown feeder. The pressure shall not vary more than 10% from said voltage, momentary fluctuations excepted.

"The Company shall monthly pay the Commission at the rate of $6.25 per thousand kilowatt-hours for all electricity delivered during the Specified Periods and for electricity purchased during hours other than those in the Specified Periods such price as shall be agreed upon by the parties from time to time.

"Prices to be paid for electricity taken shall be subject to review and

determination by the Department of Public Utilities of the Commonwealth of Massachusetts."

In estimating the revenue to be expected from the power plant under normal operation, the price for power generated during hours other than those in the "Specified Periods"—secondary power, so-called—has been assumed to be $0.003 per kilowatt-hour.

Water Available for Development of Power

For many years the yield of the Quabbin Reservoir system, including the Ware River diversions at Coldbrook, will exceed the water required from it for water-supply purposes. This excess can be drawn in whole or in part through the Quabbin Aqueduct to Wachusett Reservoir or it can be discharged into the Swift River. If sent to Wachusett, it would, for the most part, eventually find its way into the Sudbury River, after passing through a waterwheel to be built at Quabbin Aqueduct Outlet Works and existing power plants at Wachusett Dam and Sudbury Dam. The combined head of these plants amounts to about 260 ft and the power which could be thus developed would nearly double that which the same water would produce if discharged to the Swift River through the power plant at Winsor Dam, which will operate under a head of about 135 ft.

However, to draw the excess to Wachusett would, in effect, be diversion solely for the purpose of developing power, for which there appears to be no legal authorization, and the estimates of power to be realized in future years, therefore, are based on the assumption that water will be drawn from Quabbin to Wachusett only to the extent required for water supply, except such waste as may occasionally occur from Wachusett Reservoir under a reasonable system of operation of the two reservoirs in tandem.

The dams of Quabbin Reservoir are protected by a spillway about 400 ft in length at elevation 530, including a depressed section 30 ft long with a crest at elevation 528, where logs are used to regulate the crest height between elevations 528 and 530. It is possible to operate so as to provide for very large storage in the upper 2 ft of the reservoir, amounting to about 16 billion gallons.

While such operation will be made with primary consideration for flood-water storage, this provision is advantageous to the power-

plant operation, in that such storage can be released over long periods of time with a minimum waste of water that would otherwise be spilled and not available for the generation of power. Through this arrangement the periods will be lengthened when the power plant can be operated continuously at full capacity.

An emergency spillway has also been provided at a depressed section of the reservoir rim at elevation 536, which would discharge to the Swift River, should the water in the reservoir ever reach that height.

Since the station has only recently been put into test operation, there are no figures yet available on power developed or on earnings. The following data used in the design of the plant show what operating results are expected.

The average yield of the Quabbin Reservoir system, including Ware diversions, after allowing for evaporation and unavoidable spillway waste, is taken at 195 mgd or 302 cfs. This amount can be drawn continually from Quabbin without depressing the level excessively at any time. The quantity of water to be drawn from Quabbin to Wachusett for water-supply purposes will increase, as the demand of the Metropolitan District increases in the future, and the discharge to the Swift River will correspondingly decrease.

Estimates have been made of what these relative amounts may be expected to be, based upon the recorded Wachusett yields since 1928 and upon the assumption that water will be drawn from Quabbin to Wachusett, when and as required to maintain Wachusett at elevation 390 (five feet below top of spillway), and that the average yearly draft from Wachusett from all sources excepting spillway waste will be as shown. These estimates follow:

WATER AVAILABLE FOR POWER—AVERAGE MGD

	1947	*1951*	*1958*	*1963*
Draft from Wachusett	180	200	240	280
Yield from Quabbin system	195	195	195	195
Draft, Quabbin to Wachusett	58	75	112	151
Balance to be discharged to Swift River and available for power	137	120	83	44

The hydroelectric unit will normally operate under about 135-foot head, discharge 124 cfs or 80 mgd, develop about 1,620 horsepower and deliver 1,100 kilowatts of salable power. One million gallons will produce 330 kwh of salable power.

The estimated average quantities available for power are, as previously shown:

1947	137 mgd
1951	120 "
1958	83 "
1968	44 "

Since the wheel is rated at 80 mgd, it is evident that there will be more than enough water to run it continuously, 24 hours a day, 365 days a year, until about 1958.

It is estimated that power delivered and probable gross revenue, after allowing 5% for unavoidable shutdown and for use of electricity by the Commission, will be as follows:

	1948	*1951*	*1958*	*1968*
Salable power				
Primary power—millions of kwh	4.48	4.48	4.48	4.48
Secondary power— " " "	4.68	4.68	4.68	0.59
Gross revenue				
Primary at $0.00625 per kwh	$28,000	$28,000	$28,000	$28,000
Secondary at $0.003 " "	14,000	14,000	14,000	2,000
	$42,000	$42,000	$42,000	$30,000

Electricity for the Administration Buildings, the Outlet Works and the Quabbin Hill Building is still purchased at 2,300 volts from the Central Massachusetts Electric Co. This 2,300-volt energy is received at the Administration Buildings and is carried through to the Outlet Works in an underground cable and thence by underground cable and overhead line to the Quabbin Hill Building. Transformers are located at each building for local distribution at those points. When the new plant gets into final operation, the contract with the Central Massachusetts Electric Co. will be terminated. In case of shutdown of the unit, energy will be received from the New England Power Co. through the transformers and connections provided for delivery of power to the station.

The figures of estimated gross revenue after the station is in normal operation may be expected to agree closely with the results actually obtained, since, as has been pointed out, there will be more than enough water over a period of years to permit operation of the plant continuously.

Cost of Operation

The estimated cost of the development is shown below, together with an estimate of the operating cost and net income during the next few years.

Investment			
Equipment and installation			$115,000
Gross income from sale of power			$42,000
Fixed charges			
Interest	$2,300		
Depreciation	2,600		
Taxes	3,000		
Total		$7,900	
Operation			
Superintendence	$1,200		
Labor	5,000		
Repairs and supplies	500		
Administration	2,000		
		$8,700	
			16,600
Estimated net annual income			$25,400

It is expected, therefore, that the station will pay for itself during the next five years, while ample water for power is available, and will return a very satisfactory profit even after the water-supply demand increases to the point where the releases are reduced. A saving of about $3,000 per year, the amount previously spent for electricity used at the various buildings at Quabbin Reservoir, will also result from the operation of Winsor Station and it is expected

that, as operated and maintained by the Metropolitan District Commission, it will prove to be a satisfactory phase of the District's varied operations, as the two other power plants in the system have been.

ACKNOWLEDGMENT

The writer is indebted to the following members of Construction Division of the Metropolitan District Commission for assistance in the preparation of this paper: Karl R. Kennison, Chief Engineer, Walton H. Sears, Mechanical Engineer, and Albert Pearson, Electrical Engineer.

DISCUSSION

WILLIAM W. BRUSH (New York, N. Y.). I would like to ask Mr. Ginder if there is any difficulty in getting the New England Power Co. to take the current. Were they anxious to take it or was it necessary to develop an interest in taking it?

CHESTER J. GINDER. I think they were glad to have that extra power. The station installation has taken longer than it might at other times, because of war conditions, and the power, I guess, was welcome.

MR. BRUSH. I asked that question because in New York, with the Ashokan Reservoir, we tried to get the New York Edison Co. interested and they said they were only interested in selling power, not in buying it. We did, in fact, sell the power locally.

MR. GINDER. They also take the power generated at the other two stations in the system.

ARTHUR L. SHAW (Boston, Mass.). Mr. Ginder, you stated, I believe, that about 1958 you estimated that you might have only primary power, as a rule. Is it possible so to regulate your releases that they will occur only during the primary hours, or does the provision requiring releases to riparian owners downstream require you to let them out during the twenty-four hours?

MR. GINDER. Well, I believe that that can be adjusted pretty well to take advantage of the primary-hour period. Of course, we do have eight square miles between the dam and the point at Bondsville where it is measured, which does not contribute a whole lot.

MR. SHAW. Do you plan to have attendants at this power sta-

tion all the time? You mentioned the fact that automatic devices were pretty thoroughly provided, but are you planning to have an operator there at all times when the station is running?

MR. GINDER. The station is close to the Administration Building, where our maintenance force on other work is present at all times. I believe that there will not be supervision all the time while it is running—not an operator there all the time.

MR. SHAW. Your device for giving the alarm will be stationed in the Administration Building?

MR. GINDER. Yes, the remote-control panel will have those devices on it.

MR. BRUSH. In case of the breakage of any part of the piping in the station or in the chamber where the wheel is located, would that immediately result in shutting down the flow or would that have to be taken care of by action of one of the operators?

MR. GINDER. Of course, the alarm would be sounded and I believe it would immediately start to shut down the flow through those penstock valves.

A MEMBER. Are the figures for the gross revenue, which you have estimated, based on your contract relative to your period of operation?

MR. GINDER. Yes. The contract for the primary power with the New England Power Co. has been executed. The price that we receive for the secondary power is not included in the contract or set, and the estimated revenue from the secondary power is based on the assumption of our being able to get three mills.

GROUND-WATER CONDITIONS IN THE VICINITY OF MASHAPAUG POND, PROVIDENCE, RHODE ISLAND*

BY RUSSELL M. JEFFORDS AND WILLIAM B. ALLEN**

[*Read September 18, 1947.*]

AN investigation of the ground-water resources of Rhode Island has been in progress since the fall of 1944 by the Geological Survey, U. S. Department of the Interior, in coöperation with the Rhode Island Industrial Commission. Reports have been issued covering the well data and available information on ground-water conditions in Providence (references 1, 2, at end of paper), and detailed investigations of the geology and ground-water conditions in northeastern Rhode Island have been completed (3). Fluctuations in ground-water levels throughout the State are being recorded and water-level observations were made in 33 wells during 1946. Relatively severe salt-water contamination problems in Providence are being studied also, and determinations of chloride are made monthly in three wells, to obtain data on possible expansion of the contaminated areas. Local intensive investigations were undertaken in other critical areas, such as north-central Cranston, Goddard Park in Warwick, Davisville and Exeter, to assist in the solution of specific problems of public and industrial water supply. These systematic investigations are continuing and additional reports will be issued as the studies are completed.

In order to obtain additional information on the water-bearing properties of the unconsolidated sand and gravel in the vicinity of Providence, a pumping test was made at wells of the Gorham Manufacturing Co. in the Elmwood district. The data that were obtained do not necessarily represent the conditions outside the limits of the well field, but the general methods for obtaining quantitative data may be useful to engineers, well drillers and others in New England, who are concerned with the production of water from wells.

*Published with the approval of the Director, U. S. Geological Survey, and the Rhode Island Industrial Commission.

**U. S. Geological Survey, 1405 Industrial Trust Bldg., Providence, R. I.

Recent developments in quantitative studies of ground-water reservoirs (4, 5, 6, 7) furnish reliable and relatively rapid methods for determining the capacity of aquifers to transmit water and to yield water to wells. The hydrologic characters of water-bearing formations are expressed generally in terms of the coefficient of transmissibility, or discharge per unit normal width per unit hydraulic gradient, and the coefficient of storage, or the volume of water that is released from storage in a vertical prism of the aquifer of unit cross-section under a unit decline of head. The coefficient of transmissibility of an aquifer is equivalent to the product of the saturated thickness and the permeability, and is expressed commonly in gallons per day per foot. The coefficient of storage is expressed as a decimal fraction.

Quantitative studies are indispensable to a thorough ground-water investigation, inasmuch as reasonably accurate estimates of the safe yield of subterranean reservoirs, interference of wells, and other important problems are dependent upon a knowledge of these hydrologic properties of the aquifer. The interpretation of quantitative studies depends importantly, however, upon the character and distribution of the water-bearing material and on other geologic data.

Grateful acknowledgment is made to T. W. Goodwin, Chief Engineer, and W. Harris, Engineer, of the Gorham Manufacturing Co., Providence, for coöperation and assistance in obtaining data on the wells and for modifying operating procedure so as to facilitate the pumping test. Appreciation is expressed, also, to colleagues in the Geological Survey who have reviewed this manuscript.

Description of the Well Field

The Gorham well field is located along the eastern side of Mashapaug Pond in southwestern Providence (Fig. 1). The ground-water supply of the Gorham Manufacturing Co. is at present obtained from two drilled wells, Pro. 49 and Pro. 50,[1] located on a narrow peninsula extending northward into the pond (Fig. 2). These wells are operated at total capacity, when the plant is active, but the rates of withdrawal are reduced considerably at night and during week-ends. This water is used for drinking in the plant and for cool-

[1]Well numbers are those used in the reports on the Providence area (1 and 2).

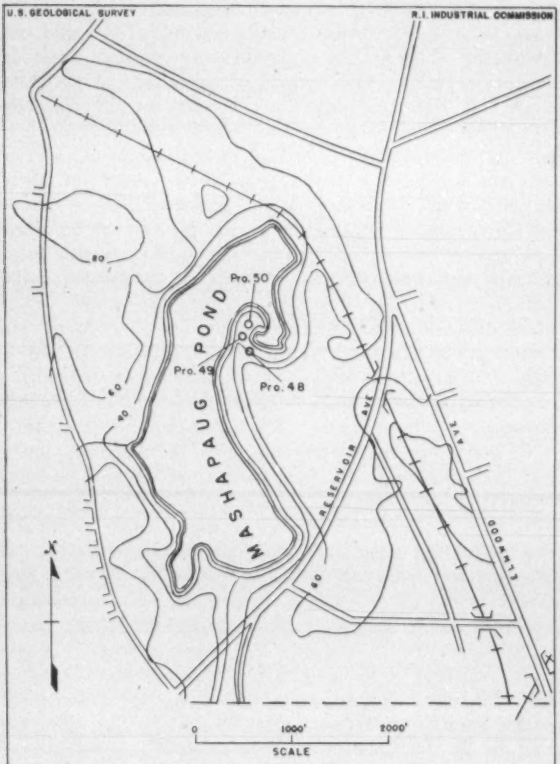

FIG. I. PLAN OF THE GORHAM WELL FIELD SHOWING LOCATION OF WELLS IN THE VICINITY OF MASHAPAUG POND, PROVIDENCE, R.I.

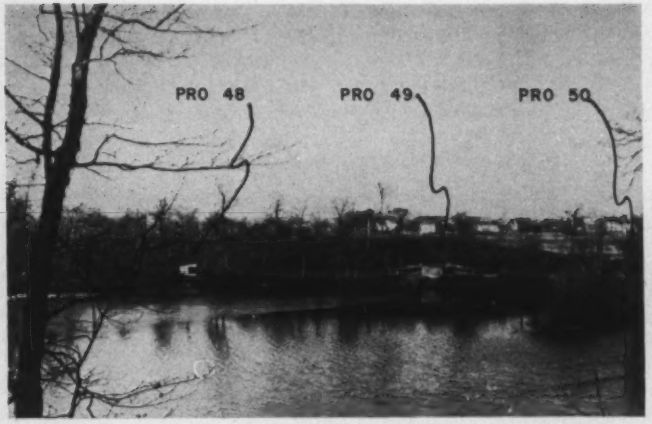

FIG. 2.—VIEW LOOKING WEST, SHOWING THE THREE WELL SHELTERS ALONG THE PENINSULA INTO MASHAPAUG POND.

ing; a separate supply from Mashapaug Pond serves the sanitary system.

Well Pro. 50 is a screened 8-inch well, that is 106 ft deep and has a maximum yield of about 110 gpm. Well Pro. 49 is a screened 8-inch well, in which the screen is placed between 100 and 108 ft below the surface; the maximum yield averages 250 gpm. Well Pro. 48 is an unused 8-inch well that is 121 ft deep. These wells were installed in 1939 and 1940, to supplant a system of ten 3-inch gang wells that were driven in 1894 to an average depth of 120 ft, in the area to the south of the present wells.

The sediments underlying the well field comprise stratified glacial outwash sand, gravel, and silty clay. Two distinct layers, an upper silty gravel and a lower bed of sand, are exposed along the hillside a few feet above the level of the wells. The size of the component grains and the sorting of these beds are shown in Table 1.

The relatively few data that are available on the material encountered in the wells at the Gorham well field indicate that fine sand

TABLE 1.—MECHANICAL ANALYSIS OF GLACIAL OUTWASH MATERIALS EXPOSED IN THE GORHAM WELL FIELD (RESULTS IN PER CENT BY WEIGHT).

Sample	Coarse gravel (larger than 8.0 mm)	Medium gravel (8.0 to 4.0 mm)	Fine gravel (4.0 to 2.0 mm)	Very coarse sand (2.0 to 1.0 mm)	Coarse sand (1.0 to 0.5 mm)	Medium sand (0.5 to 0.25 mm)	Fine sand (0.25 to 0.125 mm)	Very fine sand (0.125 to 0.062 mm)	Silt and clay (less than 0.062 mm)
A	——	1.53	0.14	1.53	2.80	3.17	1.30	8.50	81.03
B	3.80	9.70	1.57	9.01	15.81	25.11	19.20	5.86	9.94

A. Silty sand layer exposed a few feet above wells in bank.
B. Silty gravel layer exposed immediately above the silty sand in bank.

and silt layers occur to a depth of about 100 ft. The water-bearing material consists of moderately coarse gravels immediately between the base of the silty beds and the top of the consolidated Carboniferous bedrock formations. Test drilling to the north and south of the present wells indicates that silty clay reaches a depth of approximately 110 ft and is underlain by about 40 ft of intercalated gravel and clay.

According to the available data, the Gorham well field seems to lie along the eastern boundary of a buried valley, that has been traced from the Pawtuxet River in Cranston northward under the city of Providence. Relatively thick deposits of water-bearing gravel occur along the western side of the valley, as in Cranston about 0.7 mile west of the Gorham well field.

The hardness of the water pumped from the 3-inch wells along the southern part of the well field was reported as about 90 ppm and the chloride as about 18 ppm. The concentration of iron was relatively low in the 8-inch wells when first drilled, but has increased considerably. In 1945 the iron was 3.8 ppm in well Pro. 50 and more than 30 ppm in well Pro. 48.

Observations of water-level fluctuations in the unused well Pro. 48 have been made by the U. S. Geological Survey since December,

1944, and a continuous water-level recorder has been maintained on the well since June, 1945. Although the water levels vary markedly each day in response to variations in pumpage, the graph of the lowest weekly level (Fig. 3) indicates that recharge to the water-bearing beds is adequate to replace the quantity of water withdrawn during recent years.

Pumping Test

A controlled pumping test was made on March 15, 1947, well Pro. 49 being used as the pumped well and well Pro. 48 as the observation well. Well Pro. 50 was not in operation during the test and there were no openings in the pump on wells Pro. 49 and 50, through which water levels could be measured. All wells were shut down for about 15 hours preceding the test and the water levels probably returned to static level. Well Pro. 49 was pumped at an approximately uniform rate of 254 gpm for 90 minutes, and measurements of water level in the observation well were made by the wetted-tape method at brief intervals during the period of pumping and for 114 minutes after the well was shut off, to determine the rate of recovery. During the pumping period, 78 measurements of water level were made, and 51 measurements were made during the subsequent recovery (Fig. 4). The temperature of the water was 53° F. throughout the test. The water flow during the pumping test is shown in Fig. 6.

Quantitative Determinations

An equation developed by Theis (4) gives the drawdown in the vicinity of a pumped well in terms of the discharge of the well and the coefficients of transmissibility and storage of the water-bearing formation:

$$s = \left(\frac{Q}{4\pi T}\right) W(u)$$

The term s is the drawdown produced by a continuous uniform withdrawal of water from an aquifer of wide extent and uniform thickness and permeability; Q is the discharge of the well; T is the coefficient of transmissibility; and $W(u)$ is the well function of u or the negative exponential integral of $-u$. The value of u is equal to $\frac{r^2S}{4Tt}$,

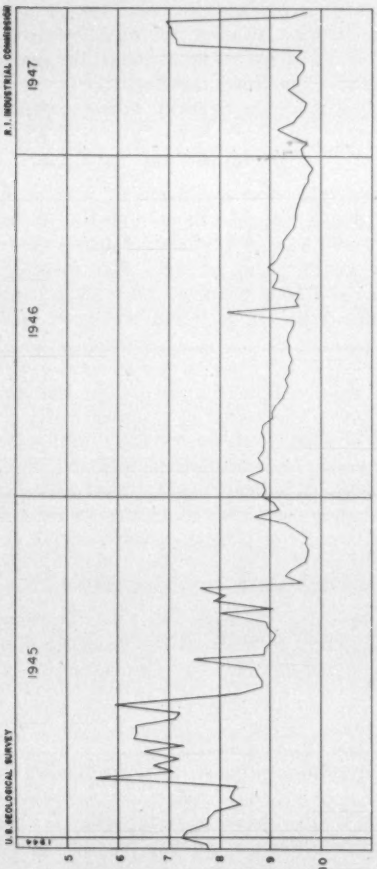

FIG. 3. HYDROGRAPH SHOWING THE LOWEST OBSERVED WEEKLY WATER LEVEL IN WELL PRO. 48. MEASUREMENTS FROM CONTINUOUS WATER-LEVEL RECORDER SINCE JUNE 1945.

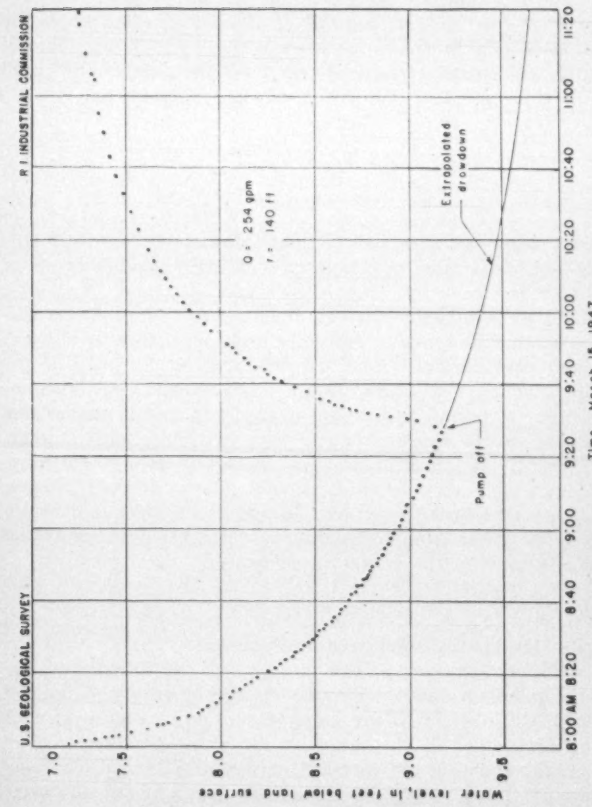

FIG. 4. GRAPH SHOWING OBSERVED WATER LEVELS IN WELL PRO. 48 DURING THE PERIOD OF PUMPING AND RECOVERY, AND THE EXTRAPOLATED DRAWDOWN FOR THE PERIOD OF RECOVERY

where r is the distance from the discharging well, S is the coefficient of storage, and t is the time since the start of withdrawal.

For artesian conditions in bedrock or alluvial aquifers, where the water level rises in wells above the level of the water-bearing formation, and for water-table conditions when the distance (r) is small and the time (t) is great, this equation may be simplified to

$$T = \frac{263.52Q}{\Delta s} \text{ and}$$

$$S = \frac{2.1 \times 10^{-4} T t_0}{r^2}$$

by using the following units and method. Relatively thorough discussions of the derivation, application, and other modifications of these formulae are given by Cooper and Jacob (6) and Jacob (7). It is pertinent to point out, however, that the formula assumes discharge at a steady rate from an extensive artesian aquifer of uniform thickness and permeability. The artesian aquifers in some of the buried valleys of New England seem to satisfy the basic assumptions sufficiently to permit utilization of the formula for engineering approximations of the coefficients of transmissibility and storage. Precise quantitative application of these values to solution of long-term problems, such as safe yield, is particularly difficult because of the limited extent of the aquifers. Where the aquifer is partially dewatered under water-table conditions, an adjusted drawdown (s_0) must be used instead of the actual drawdown (s).

$$s_0 = s - \left(\frac{s^2}{2m}\right), \text{ where}$$

m is the initial thickness of the saturated aquifer.

The coefficient of transmissibility (T) is in gallons a day per foot; Q is in gallons a minute; s is the change in drawdown in feet over one logarithmic cycle on the graph described in Figure 5; t_0 is the time in minutes at the intercept of the zero-drawdown line on the same graph, and r is the distance in feet from the discharging well to the observation well. The drawdown, which is the difference between the water level during or after pumping and the original static water level, is plotted against the time since start of pumping, in minutes, on semi-logarithmic paper with time on the logarithmic scale.

These calculations assume that the pumping rate is essentially uniform and uninterrupted, water-level measurements are made frequently with an accuracy of at least 0.1 ft, the pumping well penetrates the aquifer completely, and the time at which withdrawal began and the times at which water-level measurements are made are recorded accurately.

Drawdown test.—Water-level measurements during the period of pumping are shown on Figure 4 and drawdowns at selected times are given in Table 2. These drawdown data are plotted against the

TABLE 2.—OBSERVATIONS ON WELL PRO. 48 DURING DRAWDOWN TEST, USED IN FIGURE 5. (STATIC WATER LEVEL BEFORE PUMPING BEGAN WAS 7.04 FT BELOW THE MEASURING POINT.)

Time (t) in minutes	Water level in feet below m.p.	Drawdown (s) in feet
2.5	7.42	0.38
5	7.62	0.58
10	7.89	0.85
15	8.07	1.03
20	8.23	1.19
30	8.48	1.44
40	8.67	1.63
50	8.81	1.77
60	8.94	1.90
70	9.03	1.99
80	9.12	2.08
90	9.19	2.15

time (t) in minutes on semi-logarithmic paper with t on the logarithmic scale (Fig. 5). A straight line is drawn through the plotted points and Δs, the difference in drawdown over one logarithmic cycle, is determined as 1.45 ft (2.22 — 0.77 = 1.45). Therefore,

$$T = \frac{263.52Q}{\Delta s} = \frac{263.52 \times 254}{1.45} = 46{,}200 \text{ gpd/ft.}$$

Some of the points during the early part of these tests do not fall along a straight line; these points, therefore, cannot be used in obtaining the hydrologic constants.

In determining the coefficient of storage, the straight line through the plotted points (Fig. 5) is extended to the zero-drawdown line and values of t_0 are read at this intersection. Therefore,

$$S = \frac{2.1 \times 10^{-4} T t_0}{r^2} = \frac{2.1 \times 10^{-4} \times 46{,}200 \times 2.98}{140^2} = 1.5 \times 10^{-3}.$$

Recovery test.—Similar calculations may be based on the water-level measurements taken during recovery (Table 3). The observed

TABLE 3. OBSERVATIONS ON WELL PRO. 48 DURING RECOVERY TEST, USED IN FIGURE 5.

Time (t') in minutes	Actual water level in feet	Extrapolated water level in feet	Recovery (s') in feet
2.5	8.74	9.21	0.47
5	8.60	9.23	0.63
10	8.36	9.25	0.89
15	8.19	9.28	1.09
20	8.06	9.32	1.26
30	7.85	9.36	1.51
40	7.70	9.42	1.72
50	7.60	9.46	1.86
60	7.51	9.50	1.99
70	7.45	9.54	2.09
80	7.38	9.57	2.19
90	7.33	9.60	2.27
100	7.28	9.62	2.34
110	7.25	9.63	2.38

water levels during the period of pumping and the following recovery period are plotted against time, and the decline in water levels is extrapolated beyond the period of pumpage to correspond to the probable continued decline, if withdrawal had been continued (Fig. 5). The distance between this extended drawdown curve and the observed recovery curve at selected times comprises recovery that is equivalent to the drawdown (s) used above. These recovery values are plotted on the semi-logarithmic paper against the time (t') since pumping ceased, and a straight line is constructed again through the plotted points. By means of the same formulae, values of the coefficient of

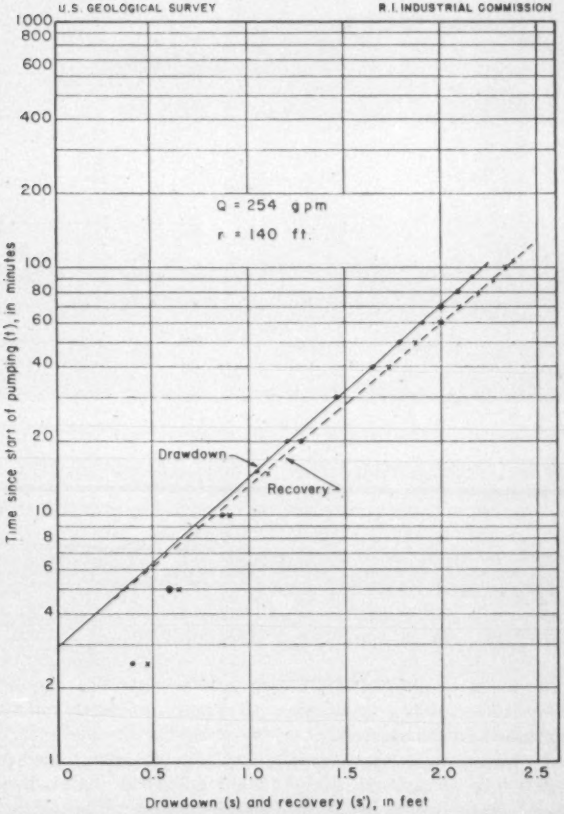

FIG. 5 TIME-DRAWDOWN GRAPH FOR OBSERVATION WELL PRO. 48, WHICH IS 140 FEET FROM THE DISCHARGING WELL

FIG. 6.—VIEW SHOWING WATER FLOW DURING THE PUMPING TEST.

storage and transmissibility may be determined to check with the values computed from the drawdown data.

$$T = \frac{263.52 \times Q}{\Delta s'} = \frac{263.52 \times 254}{1.53} = 43{,}800 \text{ gpd/ft.}$$

$$S = \frac{2.1 \times 10^{-4} T t'_0}{r^2} = \frac{2.1 \times 10^{-4} \times 43{,}800 \times 2.95}{140^2} = 1.4 \times 10^{-3}.$$

ANALYSIS OF DATA

The values of the coefficients of transmissibility and storage, as determined from observed water levels during the period of pumping and recovery, are relatively similar, and indicate a coefficient of transmissibility of approximately 45,000 gpd/ft and a coefficient of storage of about 1.4×10^{-3} for the well tested. The permeability cannot be calculated, as the thickness of the water-bearing material is not known accurately. The regular increase in drawdown proportional to the logarithm of the time since the discharge began (Fig. 5)

precludes appreciable recharge from the nearby pond in the vicinity of the well, but such recharge may occur some distance away. The low value of the coefficient of storage shows, moreover, that the water is under artesian pressure, probably due to confinement by intercalated beds of clay and silt.

On the basis of the determinations of the hydrologic character of the aquifer, the theoretical drawdowns in the vicinity of the pumped well for any particular rate of discharge may be calculated by substitution of appropriate values in the unmodified Theis equation. This formula, adjusted for convenient units, is

$$s = \frac{114.6Q}{T}\left[-Ei\frac{(-1.87r^2S)}{Tt}\right],$$

where s is the drawdown in feet, Q is the discharge in gallons a minute, T is the coefficient of transmissibility in gallons per day per foot, t is the time since start of discharge, r is the distance, in feet, from the discharging well to the place where the drawdown is to be determined, and S is the coefficient of storage. Values of the exponential integral are available in papers by Kazmann (8) and Wenzel (5).

By substituting the determined values of transmissibility and storage in this equation, curves may be constructed showing the drawdown that any desired rate of withdrawal would cause at specific distances from the pumped well during continuous withdrawal, or the drawdowns along a line extending away from the pumped well after a specific period of time (Figures 7 and 8). These curves comprise straight lines on semi-logarithmic graphs, where time is plotted on the logarithmic scale, but their significance is more evident when arithmetic coördinates are used.

Although these theoretical data assume continuous operation at a uniform rate and an extensive aquifer of uniform thickness and permeability—conditions which are rarely if ever satisfied—the determinations are generally of value in making reliable estimates as to desirable rates of withdrawal, well spacing, and probable interference of wells (9). For example, Figure 7 indicates that the drawdown after 1 day of withdrawal at the rate of 250 gpm would be approximately 4.3 ft at a distance of 100 ft from the pumped well, and 3.4 ft at a distance of 200 ft. Therefore, the total yield of wells separated

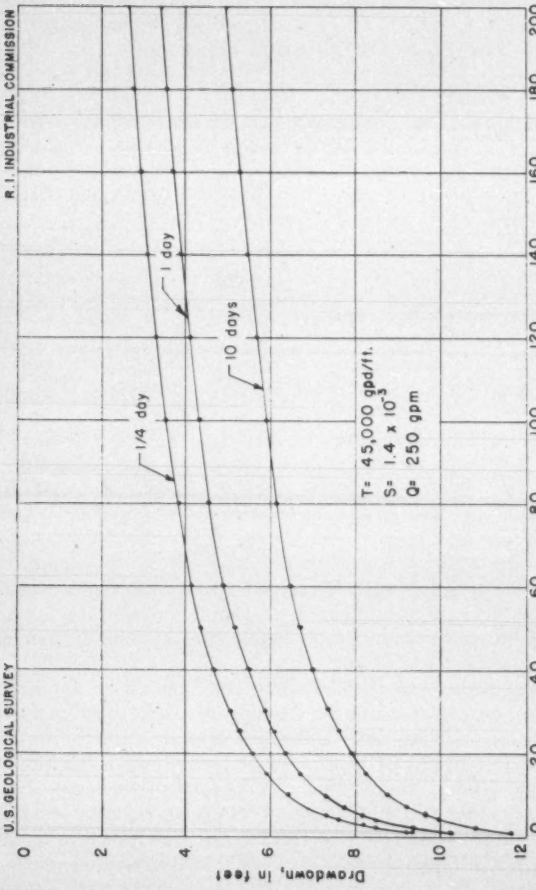

FIG. 7. THEORETICAL DRAWDOWN IN AN INFINITE AQUIFER AT THE END OF 1/4, 1, AND 10 DAYS AS COMPUTED FROM THE THEIS NON-EQUILIBRIUM FORMULA

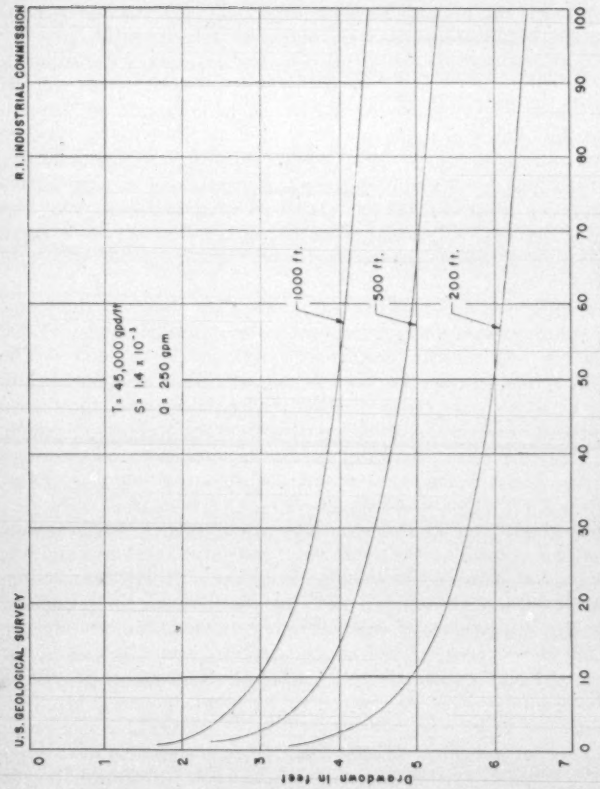

FIG. 8. THEORETICAL DRAWDOWN AT SELECTED DISTANCES FROM THE PUMPED WELL AS COMPUTED FROM THE THEIS NON-EQUILIBRIUM FORMULA

by only a few hundred feet in this area would be substantially lower when operated together than when the wells are pumped independently. A much more comprehensive investigation would be needed to determine reliably the safe yield of the area as a whole.

Summary

This paper presents the results of an investigation of ground-water conditions in the Gorham well field near Mashapaug Pond in southwestern Providence, R. I. Geological reconnaissance indicates that the well field is located along the eastern edge of a large buried valley and the thickness of the unconsolidated deposits thins importantly to the east. The water-bearing material comprises a small to moderate thickness of gravel that is overlain by about 100 ft of clay and silty sand.

A pumping test was made and, from the data obtained, the coefficient of transmissibility was computed to be approximately 45,000 gal a day per foot and the coefficient of storage 1.4×10^{-3}. The application of the straight-line solution of the Theis non-equilibrium equation to these pumping-test data is described, and theoretical curves for drawdown in the vicinity of a pumped well are included. The safe yield of the area cannot be computed without many additional data on the limits of the aquifer and the places at which recharge occurs.

The geologic and hydrologic information relate specifically to the immediate vicinity of the wells only, but procedures in obtaining the pumping-test data and in calculation of the observed information are applicable to other similar well installations in New Engand. More detailed descriptions of this and other methods for computation of pumping-test records, as well as comprehensive discussions of the derivation and application of the equations, are given in papers by Jacob (6, 7).

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Discussion

A Member. I would like to ask the gentleman if his studies have determined fairly definitely the extent of that sand and gravel deposit in the Cranston section—that is, the width of the valley and the probable depth of the aquifer itself.

William B. Allen. The only information we have is at the Narragansett Brewing Company's wells in the Mashapaug Pond area. They are only separated by a quarter of a mile. The valley runs north and south there, so that we really have a section across the valley at that point. We find that the water-bearing beds are thicker and more permeable on the west than on the east.

The Member. What about the range of thickness, minimum and maximum, in those beds?

Mr. Allen. On the west side the beds are permeable all the way down in certain places.

The Member. To what depth?

Mr. Allen. To a depth of about 120 ft. We do not have the middle of the valley. One side of the valley goes to at least 120 ft.

Warren J. Scott (Hartford, Conn.). Do I understand that

the only observation you made during that pumping test was the rate of discharge from that one well?

MR. ALLEN. Yes. We only used the one well for this test.

MR. SCOTT. The rest of the calculations were all based on theoretical knowledge of the aquifers?

MR. ALLEN. Well, no. The information we got from that one pumping test, the water-level measurements, enabled us to use these formulae that were developed.

MR. SCOTT. Just with the one well?

MR. ALLEN. Just with the one pumping, yes, sir. We were pumping one of the wells and taking the observations of one of the other wells nearby.

MR. SCOTT. A 90-minute pumping test?

MR. ALLEN. A 90-minute pumping test, yes, sir. You see, the aquifer was under artesian conditions and we could use a much shorter test. If it had been permeable all the way down, we would have had to make the test for at least a week, or three or four days anyway.

ARTHUR L. SHAW (Boston, Mass.). You had made test borings, however, to determine the general character of the underlying material, independent of these wells, had you not?

MR. ALLEN. No, we made no test borings, but they were made. That is, samples were taken when the wells were drilled, so that we had that information available.

WILLIAM W. BRUSH (New York, N. Y.). Is there any way of estimating the probable accuracy of the computations, other than by ultimately carrying on a pumping test over a long period of time, that would indicate the actual yield that could be obtained from that aquifer? Have you any estimate that it is 50% accurate, 25% accurate or 100% accurate?

MR. ALLEN. This is the first one we have made in Rhode Island. Maybe Mr. Brashears, who was in charge of the ground-water work in New England, would have more information on that in other areas where he has worked.

M. L. BRASHEARS, JR. (Jamaica, N. Y.). The best checks that I can recall, Mr. Brush, are a number of long pumping tests that we made out in the Middle West, in which permeability was included. The field result, using these various equations, gave certain figures,

which later were checked by making laboratory determinations of the permeability of materials, through which the water passed and from which it was pumped, and we found that there was a pretty fair degree of check between the two. I do not recall what the percentage would be but I seem to remember some tests in Nebraska, in which the permeability by the field method was calculated to be somewhere around 1,000 or 1,100 and the laboratory method showed the permeability to be in the order of 900. That was a pretty good check in that particular territory but it would all depend, of course, on your actual field conditions. As Mr. Allen has said, we assume an infinite aquifer under homogeneous conditions and a number of other things which can never be met in the field. These methods are designed to give the best engineering approximation that is possible. Of course, I assume that could be checked by putting down a lot of test wells.

A MEMBER. I was interested about the relation of that surface pond to the pumped wells. I wondered if there were any long-time water levels of the pond available and, secondly, whether the chemical analyses of the pond and the ground water showed great similarity.

MR. ALLEN. There have been no measurements made of the water levels in the pond that I know of, and the pond is more or less polluted by surface contamination and mills, so that no available information is to be had on the chemical content of the water. We have available only those chemical constituents that I spoke about in the paper.

THE MEMBER. For instance, was the ground water different in hardness, or in some things like that, from the surface water? Was it considerably harder?

MR. ALLEN. In general, the surface water in Rhode Island is very soft and the ground water is harder. What usually happens is that the surface water is very soft, the water in the outwash deposits—the till areas—is still harder, and the water in bedrock is even harder than that. But it is hard to state any definite figures on those.

MR. BRUSH. You said the iron content went up rather rapidly to a rather high point. Did you have any determination as to the probable cause of that increase in the iron?

MR. ALLEN. Yes. I don't know whether I should tell it or not. There is an area in here, where some time ago they dumped a lot of old scrap iron, and perhaps that had something to do with it.

A. LELAND GLIDDEN (New York, N. Y.). I would like to know if you took any bacteriological tests of your pumped water.

MR. ALLEN. No, there were no bacteriological tests made of the pumped water.

MR. GLIDDEN. Or chemical?

MR. ALLEN. Only those that I mentioned.

MR. GLIDDEN. There was no relationship between the water pumped out of this well and the water in the pond, as far as you could tell from the tests that you did make?

MR. ALLEN. No. All the water that is used for sanitary purposes is put back into the pond—that is, it is really polluted water—so that we did not bother to do anything with the pond water at all.

MR. GLIDDEN. You mentioned that one well in that area was used for drinking water.

MR. ALLEN. That is right.

MR. GLIDDEN. How far was that from the well that you used for test purposes?

MR. ALLEN. They pumped two wells, Providence 49 and Providence 50. Our observations were made in an unused well nearby, and the iron content and the other chemical constituents were fairly low in those two Providence wells.

MR. GLIDDEN. The well that is used for drinking water is one of the wells that you tested?

MR. ALLEN. Yes.

MR. GLIDDEN. I am assuming that that had been tested by the State before they would allow them to use it for drinking water.

MR. ALLEN. I am sure that it has.

MR. SHAW. I think that the last two speakers—if I understand them correctly—are trying to clear up the statement that you made, Mr. Allen, in your paper, that you had indications that there was no recharge from the pond itself—that the recharge, if it occurred at all, was from some distance away. I think they are trying to find out on what data you based that statement.

MR. ALLEN. A part of the data was based on temperatures. The

temperature of the surface water would be much higher than that of the ground water. The surface water would be around 60° or higher and the ground water was down to 53°. That is one of the things on which we based it.

MR. SHAW. I think one of the important things to note, as has already been brought out, is that these formulae are based on certain assumptions, such as an infinite layer of uniform material, which, while it may be approximately true in some parts of the country, certainly is not true in New England. I expect this U.S.G.S. team that is working in New England is gradually and effectively building up information for us. If they have not worked in New England before, they are probably finding out that our subsurface materials are far from infinite, far from being of a uniform character, and far from being uniform in permeability. Nevertheless, the formulae do point the way to these things and are certainly going to be extremely helpful in interpreting our result. But I think we are likely to find from actual experience that our accuracies, as Mr. Brush has intimated, are more apt to be on the order of 50% than the 90% that you found.

MR. BRASHEARS. I would like to make a remark on the question that these two gentlemen had. There are two indications that not much of the water in the pond was getting into the wells or, rather, that it was getting down there by a very circuitous route. One is that coefficient of storage, of which Mr. Allen spoke. If much water got down there—if the formation were not rather good artesian formation—that figure would approximate water-table conditions and would be much higher. It would approximate the specific yield of the water-table bed.

The other indication would be the drawdown curve that he showed you. The one he showed was a very smooth curve. If he began to get a sizable amount of water from the pond, that would constitute another source of recharge and his curve would flatten very definitely. As it came down, it would lose its smooth shape and flatten.

Since he does not have either of those things, it is a pretty good assumption, based on other tests, that the pond water that is coming into that well is going way around this clay, perhaps to the

other side of the valley, if you wish to visualize it that far, before it can get under.

MR. SHAW. I wonder if Mr. Howard would care to say anything. He has had a lot of experience with ground water here in New England.

PAUL F. HOWARD (Boston, Mass.). I don't think I have anything in particular to say about it. We have followed these formulas and have been trying to apply them to past records of a great many time tests. We have not had enough success with them to feel justified in using them in the future for advance opinion or as a basis for our studies. We have seen fit to stay with the long pumping tests, in which the drawdown water level is stabilized over a long period of time, observing our recovery and making calculations on that basis. As you suggested, our material lack was in uniformity in the deposits.

It is so difficult to compute the size of any area that is furnishing water. A good illustration is on Cape Cod, at Brewster, where the apparent valley is no bigger than this room, yet pumping tests showed that it yielded 300-400 gpm continuously. There are so many unknowns which have yet to be determined in applying the formulas that I have not dared to use them in making predictions of what would happen in certain places.

MR. BRUSH. I would like to ask this question. If it is embarrassing, do not answer it. In reporting your results of a test like this, is it clearly brought out in the text of the report that the estimates made by using these various formulae represent an estimate that should not be taken as the probable yield, necessarily, but as the best you can do with the information available? In other words, is ample warning given to those who do not know so much about what was done or attempted, so that they would not take those figures and use them incorrectly?

MR. ALLEN. We have tried to indicate that.

FRANCIS H. KINGSBURY (Boston, Mass.). I wonder if the government has made any tests of the wells at Camp Edwards, where, as I remember it, the conditions approach theoretical, with more or less uniform sand of a depth of some 90 ft. Then there is another one of that same general type in the Charles River wells, so called,

at Needham, where they go down the unusual distance in this State of some 90 ft and have a uniform material.

MR. SHAW. I understand that the Geological Survey is planning to extend its investigations over New England as time goes on. Have you done much in Massachusetts yet at either of these places?

MR. ALLEN. I think that Mr. Brashears can answer that better than I can.

MR. BRASHEARS. There have been no tests of this kind. We have not run tests at Camp Edwards, but we would welcome the opportunity. The conditions have not been right to do that.

MR. HOWARD. The work that they did in the valley at Lowell we found to be extremely helpful in the furnishing of new water supplies there and in determining the location of the glacial valley, and we are hoping that they will do a lot more of that in New England. I think it will be of tremendous benefit to the communities, when that work is carried forward, in helping us to locate and follow out all these valleys here in New England, as then the communities will know what they can do in the future in the way of furnishing ground-water supplies. Of course, we have extensive buried valleys with great potential value in furnishing ground water. The old, buried Sudbury River valley, I suppose, is the one that has been most developed at Framingham, Natick, Wellesley and Needham. When their studies are carried out further on the old, buried Merrimack River valley, as well as many others, we shall have a wealth of information and it may be a cinch to get a good water supply for a lot of these communities.

MR. SHAW. One point that has not been mentioned here today is that this survey, when it is complete, is very likely to furnish data which may lead to control of the ground-water sources in New England, as has been done for some time on Long Island. Today the officials of any town or city, who think that perhaps they need to increase their sources, are at perfect liberty to make tests and inspections and to put down wells wherever they want. I expect that the survey data are likely to show the trend and, when ground-water levels over the years, perhaps, show us that we are approaching the limit of what can be done with ground water, the inevitable result of that is better control—statewide control, perhaps—of the use of ground water.

FLOOD-CONTROL STRUCTURES

PART I.—DESIGN

BY JOHN E. ALLEN*

[*Read September 18, 1947.*]

IT IS a privilege to be able to speak before an organization having the background and standing of the New England Water Works Association. Although it may seem like carrying coals to Newcastle to talk to you of water, the problems involved in flood control are different from many which you customarily meet.

Disastrous floods have occurred at varying intervals on most of the large rivers in this country since Colonial days. The extent of the damages caused have increased with the growth of the population and industry in the river valleys. Although protective levees, built by the Federal Government, have been in existence on the Mississippi and Ohio Rivers for many years, it was not until the unprecedented country-wide floods of 1936 that the Congress assumed a general responsibility for flood-protective measures.

The policies outlined in the 1936 Flood Control Act, and amended and expanded by subsequent legislation, are briefly as follows:

First: It is the sense of Congress that flood control on navigable waters or their tributaries is a proper activity of the Federal Government in coöperation with the States and their political subdivisions and that the Federal Government should improve, or participate in the improvement of, such rivers or watersheds for flood-control purposes, provided the benefits resulting therefrom are greater than the cost.

Second: Federal investigations for, and construction of, flood-control improvements on waterways shall be under the jurisdiction of the War Department and under the supervision of the Chief of Engineers.

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Third: Federal activities leading to the prevention of soil erosion and water-flow retardation on watersheds shall be under the direction of the Secretary of Agriculture.

Fourth: The Secretary of War is authorized to acquire in the name of the United States title to all lands and rights-of-way required for any dam and reservoir project or channel improvement for flood control.

Fifth: Penstocks or other similar facilities, adapted to possible future use in the development of hydro-electric power, shall be installed in any dam authorized, when approved by the Secretary of War, upon the recommendation of the Chief of Engineers and the Federal Power Commission.

Sixth: Rights-of-way for local protective works shall be provided by the community affected, which shall agree to maintain and operate such works under rules established by the Secretary of War.

Seventh: The interests and rights of the States are recognized in determining the development of watersheds within their borders and in water utilization and control.

Eighth: Investigations, plans and proposals for flood control or navigation shall be made available to the States affected, and their views and recommendations shall be transmitted to Congress along with the report by the Chief of Engineers relating thereto.

Ninth: No flood-control project shall be constructed until it has been authorized by Congress and funds have been appropriated therefor.

In accordance with directives by Congress, the New England Division of the Corps of Engineers has investigated and reported upon all of the major rivers in New England. As a result of such reports, Congress has authorized flood-control projects for the Connecticut, Merrimack, Blackstone, Pawtuxet, Thames and Housatonic river basins. Insofar as agreements have been reached with the States concerned and construction funds have been made available, individual projects have been built on the Connecticut and Merrimack Rivers, which will provide a limited amount of flood protection on these streams.

The four principal watersheds in the State of Maine have also experienced great floods. However, the areas subject to inundation

have not been developed as extensively as in the other valleys mentioned and the cost of protective works would exceed the anticipated benefits. The Androscoggin River, with its numerous storage reservoirs, is the best-regulated stream in New England.

There are certain logical steps in the investigation and development of a flood-control plan. They are concerned with the questions of what has occurred, what may be expected to occur and what is the best and most economical solution. I will expand these steps in the order named.

What Has Occurred. The first step involves a study of floods of record and of the damages caused. If we had records as long as those on the Nile in Egypt, which go back for nearly 4,000 years, we could prognosticate with great accuracy the probable frequency of occurrence of floods of any magnitude. As it is, New England is more favored than most sections of the country, since we have accurate records going back 100 years for several of the rivers and hearsay data for an additional 150 years. There are several accepted methods of developing frequency curves from such flood records as we have and the results are not in strict agreement. For this reason, I hesitate to quote frequencies. However, no better tool has been developed for converting losses of record to annual losses as a basis for economic studies. The New England Division uses probability equation $Y = \frac{m}{n - 0.5}$ for plotting the discharges of record, where Y equals the probability of occurrences in years, m is the period of historical record, and n is the order of magnitude of the flood. The upper limit of the discharge-frequency relation is fixed by the estimated discharge of the maximum possible flood.

The damages to be anticipated from a flood of any magnitude can be obtained with reasonable exactness, particularly if data are available for one or more major floods. The discharges which took place in 1936 were the maximum of record on many of the streams and the data secured were used as a basis of stage-damage studies. Immediately following the flood, field parties were sent out, which visited all the damaged areas. Individual losses were computed and checked with the estimates furnished by each property owner. In

addition to the losses sustained, the damages were computed for higher and lower stages, down to the point of zero damage. By summarizing these figures, stage-damage curves were formed, which gave the anticipated losses for floods of any magnitude.

At the same time that these investigations were going on, survey parties obtained the elevations of the flood profiles and established the control points, such as dams and river constrictions, which governed the flood elevations. These dams and constrictions were surveyed in order to establish rating curves and the rivers were divided into reaches, within which there was slight variation in the flood level. By applying the rating curve to each reach thus established, the water level could be determined for a flood of any magnitude.

Flood losses are made up of a variety of elements. Direct losses are the cost of replacement of actual physical damages suffered. In addition, there are indirect losses, such as loss of business or wages, cost of re-routing highway and railroad traffic around the damage areas, flood fighting and relief of distress.

For convenience, the direct losses have been broken down into the following categories:

1. Rural;
2. Urban;
3. Industrial;
4. Highways;
5. Railroads.

The indirect losses vary with the type of direct damage and with the individual community. It has been found by sampling methods that a fair yardstick of the measure of indirect losses is represented by the following ratios of indirect to direct losses:

1. Rural, 20%;
2. Urban, 55%;
3. Industrial, 120%;
4. Highways, 100%;
5. Railroads, 100%.

The direct and indirect losses together constitute the flood damage. The summation of these values for each stage elevation, multi-

plied by the frequency of the flood reaching that stage, represents the average annual flood losses.

There are certain indeterminable losses not listed, such as loss of life, human suffering and impairment to health, which cannot be evaluated but which should be given consideration in the selection of a flood-control program.

In computing annual losses, it is necessary to eliminate those which are non-recurring. Thus, the loss of a highway bridge, which has been replaced with one of adequate clearance, should not be included in the determination of annual losses.

The benefits, of course, are the reductions in losses occasioned by the construction of flood-protective works.

What May Be Expected to Occur. The second phase of study deals with the magnitude of future floods which might be expected and to what degree protective works should be provided. As previously stated, the records of stream flow in this country are comparatively short and any flood of record can be easily exceeded. The watershed is first studied to determine the extent to which each major branch stream contributes to the peak flow in the main river for the floods of record. An analysis of the rainfall data for the storm in question is then made and it is usually found that, if the center of that storm had occurred a few miles away, the results would have been quite different. In conjunction with the Weather Bureau, a storm pattern is worked out, based on a storm of record and on orographic conditions of the basin, which would produce a greater flood than has actually occurred. Thus, we arrive at what is known as the project storm and the flood which it would cause is called the project flood. This project flood is one which can be easily anticipated to occur and, as a rule, is 15% to 30% greater than the flood of record. In some of the tributaries, the project flood will exceed that experienced by 50-60%. The maximum possible flood which reservoir spillways are designed to pass is greater than the project flood.

Best and Most Economical Solution. The third and final phase is the development of a flood-control plan which will provide the maximum protection within economic limits. These economic limits, as defined by Congress, are that the benefits shall be greater than the cost. There are three common methods of accomplishing flood pro-

tection: channel improvement, local dikes and reservoirs. Channel improvement is usually the most expensive method and, in some cases, the situation downstream is aggravated by channel straightening. Local dikes and river walls are feasible for large centers, such as cities, on flood plains, where the damages are intense and highly concentrated. A dike protects only the area enclosed; therefore, its scope is limited. Furthermore, provision must be made, in the form of intercepting sewers and pumping plants, for handling the surface runoff within the area and for disposing of sanitary sewage, which is normally allowed to flow into the river. Otherwise, the river flood would back up in the sewerage system and find outlets through manholes in the areas to be protected. It is desirable to provide local protection for a greater flood than is the case with reservoir control. If a dike is overtopped, the whole area is inundated, whereas inadequate reservoir control will lower the flood heights, if it does not eliminate all damage. There are certain localities, such as the mill areas of Holyoke, Lawrence, Lowell and Manchester, where the canals and water passages under the mills greatly complicate the problem of local protection. On the other hand, the cities of Springfield and Hartford are ideally adapted for dikes and river walls.

Control by a proper system of detention reservoirs provides the most widespread benefits of any method of flood protection. The optimum results are obtained, if the drainage areas tributary to the reservoirs total about 50% of the whole river basin and if the reservoirs are so distributed as to control the principal flood-producing tributaries. These reservoirs should be capable of eliminating the flooding of railroads and main highways in the valley bottoms and of protecting bridges and existing dams from failure. In the event that it is not possible wholly to eliminate flood hazards in isolated damage centers by means of reservoir control, such areas can be protected by local dikes of a lesser height. For rainfall conditions in New England, the detention reservoir should have a storage capacity of 6 to 8 in. of runoff, depending on the size of the drainage area and the runoff characteristics.

The reservoirs should be located above principal damage centers and so placed as to reduce the flood flows of the tributaries and desynchronize their contributions to the peak flow on the main stem.

Numerous studies have shown conclusively that a group of small reservoirs in the headwaters is much more costly than a few larger reservoirs farther downstream, the benefits being the same. As a rule, such small headwater projects do not yield a return on the investment. Ordinarily, the intensive development along its channel precludes the building of a flood dam on the main river stem.

A reservoir with 6 to 8 in. of tributary-runoff storage can usually hold back a flood of approximately 100-year frequency, while it is discharging about 10 % of the unrestrained peak flow. It ordinarily takes 10 days to 2 weeks to empty the reservoir completely without exceeding downstream bank capacity, after the flood threat has passed. Even though a greater flood should cause discharge over the spillway, the benefits are large, as the peak outflow is much less than the inflow and the delay in timing can be helpful.

Of course, a flood-control reservoir should be empty when a flood is imminent; otherwise its value is greatly reduced. For this reason, the Corps of Engineers does not permit power or conservation storage to encroach on flood storage in a project designed for dual purposes.

The flood discharge is usually controlled by gated conduits. When an adequate flood-forecasting system is available, the discharge can be reduced for the lesser floods without letting the storage top the spillway and, in this manner, additional downstream benefits can be realized.

In order to insure safety against overtopping of the dam, the spillway is designed to pass the maximum possible flood, as determined by joint study of the Corps of Engineers and the Weather Bureau, this flood being superimposed on a full reservoir. Thus, the overall height of the dam is determined by the flood it must store plus the spillway surcharge required to pass the maximum flood plus a freeboard of 5 ft or more, depending on the fetch of the waves in the reservoir.

With the problems described in mind, it remains to develop a system of flood protection. Many alternate sites must be investigated and their costs and carrying charges compared with the benefits, before a well-balanced system is obtained. The choice of reservoir sites is often restricted by unsatisfactory foundation conditions or by intensive development of the prospective reservoir area. In New Eng-

land, the railroads and highways usually follow the river valleys and frequently towns and good agricultural land present an insurmountable obstacle. Local opposition to a reservoir is sometimes so strong that downstream communities must be denied effective protection by that means and resort must be had to the more expensive and less effective protection afforded by dikes and flood walls.

FLOOD-CONTROL STRUCTURES

PART II.—CONSTRUCTION

BY CARL H. LOVEJOY*

[*Read September 18, 1947.*]

THE building of flood-control structures in New England by the Federal Government has been a development of the last 15 years. The real program started immediately after the hurricane and flood of September, 1938. This disaster, following so closely the flood of March, 1936, gave the final impetus to the work that resulted in the starting of many concrete flood-walls and earth dikes along the Connecticut and Merrimack Rivers before the end of 1938. In 1939 and 1940 five dams were started, all of which were completed before the war interrupted the program. Two dams have been started since the war and others are to follow.

New England flood-control construction presents about the same number of problems that are encountered on the large projects of the West. The difficulties to overcome are similar but on a smaller scale.

Mr. Allen has described the use of flood-control structures and has shown that they consist of dams, to hold back part of the flood flow of the river in the upper reaches or tributaries, and walls and dikes, to confine the flood flow as it passes through the cities where the damage may be greatest. I shall confine my remarks mostly to dams.

The dams built in New England are relatively close to large cities and do not involve building a camp for the workers. This is an advantage we have over the large dams in the West. The fact that our country is more thickly settled makes it more difficult to find locations that do not interfere with existing developments. Many dams involve rebuilding a stretch of railroad and all require expensive road relocations, and it does not seem possible to build a dam

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here without moving at least one cemetery. Flood-control dams are usually dry dams, and the only time they have water against them is when the gates are closed and the reservoir fills, to hold back flood waters for discharge later, when it is safe to do so. A typical New England flood-control dam may be described as a large earth fill with dumped rock facing spread on the slopes, from 75 to 125 ft high, across a river valley about half a mile wide, with some means of passing the normal flow of the river through the dam or around the end of the dam, gates to regulate or shut off this flow, and a spillway to take care of excess water beyond the capacity of the reservoir and prevent the dam from being overtopped.

Other dirt dams have been built in New England, of which the outstanding were water-works dams, particularly Cobble Mountain and Quabbin in Massachusetts, the Saville Dam for Hartford and, more recently, the dam now under construction at Pittsfield, Mass.

The first construction problem in a flood-control dam is to take care of the flow of the river while you are building the dam. This means that one of the earliest operations must be the building of the outlet works. If one of the ends of the dam starts from a rocky hillside, the final flow of the river will be in a rock tunnel through the hillside, with an inlet channel, gates, a gate house and an outlet channel with a stilling basin. This feature of the work will usually take one year. The New England construction season for efficient work runs from about April 1st to December 1st. Work on rock tunnels and rock cuts can be performed in the winter, as those operations can be continuous, though not too extensive, in the cold weather.

Unless the river valley is very wide and the actual stream located at one side of the valley, very little work can be done on the earth fill for the dam, until the river has been diverted into its permanent location, where it flows through the outlet works you have been constructing the first year. The rock from these excavations will have to be stockpiled for use on the rock-facing of the dam, when you start placing dirt the second year.

At the earliest time in the spring, after spring freshets have passed, the river is diverted from its old bed to the outlet works and the race begins to strip the foundation, place the impervious fill in the cutoff trench and then carry up, simultaneously, the separate im-

pervious earth core, random and pervious outer sections, and dumped rock faces of the huge earth structure to a point high enough to prevent overtopping from high flows, which unfortunately can occur at almost any month in the year. Dirt cannot be placed in freezing weather, so that it is certain that the dam must be at a safe height before quitting for the winter, to be ready for high waters in the spring.

The earth for building dams in New England is always close at hand. Impervious, random and pervious materials have been distributed and assorted by glacial action. The excavation for the dam and the spillway is generally usable, at least for random fill. The impervious is more difficut to find but, with a little exploration, a borrow pit can be established not too far away. It must contain sufficient clay to be impervious and yet not too much, so as to be unworkable. The pervious is an open sandy material, that can be used bank-run from the best sand and gravel deposit that can be found. Of course, all these materials are tested in the soils laboratory in Boston during the design stages and a field soils laboratory is part of the resident engineer set-up. The Engineer Corps is one of the pioneers in the field of soil mechanics and a firm believer in scientific control of all earth fills.

Earth dams have been built successfully by hydraulic-fill methods. Cobble Mountain, Quabbin and our Knightville Dam were all constructed this way. Dams are built in this manner, when there are large deposits of glacial material right at the site, but with all classes of material mixed together. The materials are separated into impervious, random, pervious and gravel and cobbles by hydraulic methods. The earth is carried to the dam and placed by pumping it through a hydraulic dredge line. Such dams are tricky to build and a breakdown of the pump shuts down the entire work.

Dirt-moving machinery has developed and increased in size so much in the last ten years that all of the earth dams we now contemplate in New England will be placed by hauling in the material, spreading the earth in layers and then rolling it with heavy sheepsfoot rollers.

I hate to admit it, but I am old enough to have worked on the Charles River Dam right here in Boston, that created the Charles

River Basin and keeps the tidal waters from the river. That was really the horse-and-buggy days, as dirt in those days was hauled in dump trucks drawn by horses. Much dirt was loosened by plows, and dug and loaded by hand labor instead of power shovels. The few shovels and rollers available were moved by steam and not gasoline, and there was still hand-mixed concrete for culverts. Of course, in those days labor worked 9 hours for $2.00 a day and railroad laborers in New England who had a steady year-round job worked for $1.50 per day, six days per week.

Nowadays we do most of our work by machinery. We accomplish ten times as much and build structures that it would be impossible to build without the large number of horsepower, and I don't mean animals, that is available per worker on the present-day construction job.

On dam construction, dirt excavation, which was generally accomplished by power shovels, the dirt being hauled by trucks, is more and more being accomplished by scrapers, which dig and carry 15 to 21 cu yd. If the dirt is only carried a short distance, the motive power is furnished by crawler tractors but on long hauls, the whole assembly is rubber-tire mounted and will carry these loads at 25 miles per hour. If a shovel is used, it will probably drop the dirt into bottom-dump wagons of 15-yd capacity or more, mounted on large rubber tires. This change from trucks has been occasioned by two factors: first, the trucks mire down too easily when placing dirt in 6-in. or 8-in. layers; and second, the increased capacity of the present vehicles. Even with this increased capacity, the crawlers or the huge rubber tires distribute the load so that travel is possible over clayey, impervious or sandy pervious fills, that trucks would be unable to negotiate.

Excavated rock, of course, is still carried in trucks. However, if the haul is not too long, crawler-type equipment is used, as the rough floors of rock cuts are very hard on tires.

Probably some of you remember, back in the trucking days, when heavy fills were built up for their full height by backing trucks up to the edge of the fill and end-dumping down the slope. No wonder it took some years for these fills to settle to final grade!

Present specifications of the Corps of Engineers require all fills

to be placed in layers, usually 6 or 8 in. thick, and compacted by rolling. The usual roller is the sheepsfoot roller, which compacts materials from the bottom up. The sheepsfoot roller resembles the ordinary lawn roller, with iron feet around the circumference, projecting 6 to 8 in. from the surface of the roller. It is interesting to see a tractor hauling a gang of these rollers and to notice how, on the first pass, the feet will sink into the loose dirt for their full length. By the time the sixth (usually the last) pass is made, the feet will be riding almost on top of the compacted layer of fill. Tests, of course, are made to determine the density obtained. The increased density over that of the material in its natural state, before digging from the ground, is often startling. These results are obtained by controlling the moisture in the dirt at the time of rolling. If too wet, the material has to dry before it can be rolled and, if too dry, it is sprinkled with a fire hose.

Due to the care used in placing fill, a separate payment per cu yd is made for making the fill and another for digging the dirt with which it is made. This is different from highway practice, where one payment per cu yd is made for excavation, which includes the placing of the dirt in the fill. Inexperienced contractors sometimes overlook the fact, when bidding, that it may take as much as 135,000 cu yd of excavation to make 100,000 cu yd of fill, on account of the way we compact it.

Spillway excavations are almost always in rock in New England. Unless the valley is wide enough to start building a portion of the dam the first year, there will be no place to utilize the rock from the spillway until the second year, when the stream has been diverted and the construction of the dam begins. It is possible to do the rock excavation in the spillway and stockpile the rock. This involves wasteful rehandling and is not customary in New England.

You can see how busy a contractor will be in his second year on a flood-control dam. He will be digging three kinds of dirt—impervious, random and pervious—doing rock excavation and placing all of the excavated materials. All of these different fill materials are placed so that the top of the dam is brought up with its surface approximately level. Impervious and random are compacted with sheepsfoot rollers but the sandy pervious will not compact with this

type of roller, which, for the most part, stirs up the material. The compaction is accomplished by heavy caterpillar-drawn tractors, the vibration of this type of equipment packing down the sand, if it is kept moist.

Of course, there is bound to be an appreciable amount of concrete in an earth dam. The outlet works, whether they comprise a rock tunnel through one of the abutments of the dam or a conduit under the dam, are built of concrete and the spillway always has an overflow section of concrete.

I mentioned briefly the improvements in the methods of placing earth fill. The Engineer Corps has been particularly interested in obtaining durable concrete for its flood-control structures. Its specifications are rigid and are strictly enforced. Concrete materials are thoroughly tested before being approved for use. The purpose of all tests is to determine whether the concrete, when mixed and placed, will harden into a product which will withstand the ravages of time. Paints and steel have been tested like concrete, by exposing specimens to the elements over a long period of years. We learn, from the samples which endure the best, how to manufacture the product which will last longest.

To simulate exposure tests, concrete specimens are frozen and then thawed. The Engineer Corps is now stressing a new, rigid freezing-and-thawing test for concrete. The old test took 24 hr for one cycle. It was rarely feasible to wait 200 days for results. The new test produces 11 cycles in 24 hr and you have the benefit of 300 cycles in a month. Three sets of specimens are cast and tested simultaneously: one concrete is made of the sand and coarse aggregate which is under investigation; another of a proven sand with the coarse aggregate being investigated, and another concrete is made with the sand under investigation and a proven coarse aggregate. Definite results are often shown in 150 cycles. The tests, however, are carried through to a definite conclusion in all cases.

The samples are judged, not by mere visual examination or loss in weight of disintegrated concrete, but by testing them for their sonic modulus of elasticity. This is a scientific way of testing the deterioration of the concrete, much as you might hit it with a sledge hammer, to see how hard it is and whether the hammer bounces back

with a healthy ringing sound, or the material is soft and lifeless. Actually, the transmission of vibrations through the concrete is observed somewhat as a doctor observes heart beats and makes an electro-cardiograph of your heart action. The progressive deterioration of the concrete is determined from the change in the natural frequency of vibration of the concrete and from the modulus of elasticity.

It costs a great deal of money to set up such an elaborate testing plan but it is a small proportion of the value of the structures involved. New England aggregates are tested in the Corps of Engineers laboratory at Vicksburg, Miss. They get the same attention as aggregates for use on dams 25 to 50 times as large as ours. If anyone thinks this is going to extremes, he should examine some of the concrete structures placed 20 years ago, or less, and see how some of this permanent concrete disintegrated.

Of course, materials may be perfect and the concrete poor. The bride may make a terrible cake out of good materials, because she does not know how to combine the ingredients and bake them properly. So can an inexperienced (or careless) contractor make poor concrete. We endeavor to overcome that, too, by intelligent inspection. Making concrete is an art. Somebody connected with the job must have a great pride in his work and labor conscientiously to instill this spirit and desire for craftsmanship into the whole force doing the work.

The same care is used in building earth dikes and concrete flood-walls as is expended on dams. Incidentally, the upkeep of the dams is the responsibility of the Federal Government but the local flood-protection works at the cities is turned over to them upon completion, for them to operate and maintain. Flood-control construction involves not only heavy construction of dams, railroads and highways, but also building construction, when we build gate-houses to hold the gate hoists, cranes, gasoline-electric-generator emergency units, switchboards, gages, etc. Also, when we build earth dikes and concrete flood-walls, to keep the river out of cities, we have to build pumping stations, to remove the storm water and sewage from the city, when high water occurs.

From these various points that I have touched on you can see

that flood-control construction in New England involves a great assortment of operations. The trend has been for larger and larger equipment to turn out a greater amount of work with the least amount of unskilled labor possible. In spite of the increased efficiency in the present-day methods of work, the total cost of building flood-control structures continues to rise. When it will stop, it is not for me to predict.

Discussion

William W. Brush (New York, N. Y.). I would like to ask Mr. Allen as to the operation after the dams are completed. I understood from Mr. Allen that there were rules set up for operation, and he spoke about the immediate emptying of a flood reservoir after it had received a flood. I presume that might be usual but I was somewhat curious as to some of the details on that.

John E. Allen. The flood-control dams are operated by personnel furnished and paid for by the Federal Government. Local protective works are maintained and operated by the locality where they are built, under rules established by the Federal Government.

Arthur L. Shaw (Boston, Mass.). It has been said that there are some nice jobs for elderly employees, who have had some skills in their younger days in construction. These men sometimes get established as operators and they have very little or nothing to do, except to be on hand and on call—and possibly mow a little grass—but largely to be on call so that, when the flood occurs and the flood dispatcher figures that it is time to close a certain dam or regulate certain gates, the men can be accessible by telephone for prompt action. Isn't that so, Mr. Allen?

Mr. Allen. Yes, but it is not a get-rich-quick scheme.

Carl H. Lovejoy. May I take exception to the statement that they are elderly men? They are mostly men under forty and they are not sitting around waiting for something to happen. They have a gang of laborers at the dam to do general work and to keep the machinery oiled up. They have charts that Mr. Allen worked out as to how to open and shut these gates as the water rises. They are educated men and we pay them a good salary. You should not think that they are a bunch of superannuated politicians. We have a bunch of civil service men who are good.

WARREN J. SCOTT (Hartford, Conn.). Have any of these reservoirs been constructed so as to maintain a flow in dry periods?

MR. ALLEN. No, with one exception. During the war, at the Knightville dam on the Westfield River, about 1½ in. of rainfall were stored, to facilitate manufacturing downstream. That was discontinued after the close of the war. The reservoir is normally dry.

MR. SHAW. I recall on one of the dams in the Miami Conservancy District in Ohio there was quite an elaborate bronze tablet, established at the top of the dam, in a prominent place, stating in no uncertain terms the purpose of that dam and the fact that, under no circumstances, should it ever be filled or operated as a power dam. Do you take any such precaution as that to perpetuate the idea or depend simply on the regulations that you issue from time to time?

MR. ALLEN. We depend on the regulations. Of course, we can't foretell what future generations will do but the intent is that the flood-control reservoirs will be normally empty, except in the case of a flood. On a reservoir designed for combined use, some allocation would be made for power storage and some for flood storage, and one would not be allowed to encroach on the other.

THEODORE L. BRISTOL (Ansonia, Conn.). Does the growth in these reservoirs have to be cut every year?

MR. ALLEN. No, sir, we do not cut the growth in the reservoirs, for two reasons: one is that it is very expensive, and the other is that it would spoil the appearance of the land within the reservoir area, which is leased to anybody who wants it for about the cost of the taxes. There are several hundred acres in that Franklin Falls area that are leased for farming purposes or grazing purposes today, and that land is open to the first customer who wants to lease it.

HOWARD M. TURNER (Boston, Mass.). You stated the increase in damage between 1936 and 1946, giving the example of Manchester.

MR. ALLEN. Yes.

MR. TURNER. You gave as the main reason for the large increase from $4,000,000 to $14,000,000 the fact that the mills were busy. Wasn't some part of that the change in the price scale?

MR. ALLEN. We used an increase of 50-60% on the cost of replacement of damages, but in the case of Manchester the greater part of it was due to the complete occupancy of the mills there, which were absolutely empty in 1936.

MR. TURNER. Did that ratio hold at most places?

MR. ALLEN. We have found an even higher ratio than that at Lawrence and a little lower ratio at Lowell, but that may be a fair example of what might be expected, because, as you know, the 1936 flood occurred during the depression, when a majority of the mills were idle. We have found, for example, that the benefits to be picked up have increased in greater proportion than the increase in the cost of the protective work since 1936. It is surprising but so.

MR. SHAW. When Mr. Allen mentioned that spread from $4,000,000 to $14,000,000 in the allowance for damages, he also said that they did not include the so-called non-recurring damages, such as bridges, which, when rebuilt, were rebuilt so as to be no longer subject to damage. It occurred to me that there must be many borderline cases which might be called diminishing damages. If a small producer on a stream has had his 100-year-old rock-fill dam washed out, he probably replaced it with a dam which follows modern principles and very likely will not be damaged as much by the next big flood. Also, people who have had buildings damaged many times either do not rebuild or do not use the building for the same purpose. Did you take any account of those non-recurring or diminishing damages? It would be very hard to do it on a statistical basis but you would have to make some allowance.

MR. ALLEN. An old mill-dam that is washed out is not included in the recurring damages, unless it is rebuilt, and the majority of those old mill-dams are not rebuilt. They usually serve marginal mill properties, which are on their last legs anyway. It is only on going concerns that we include the damages as recurring.

MR. SHAW. How about the hay barn on the bank of the river, which was washed away, and which, when the man rebuilds it, he moves back 200-300 ft, so that it will not be damaged?

MR. ALLEN. If it is rebuilt outside the flood zone, the damage would not be a recurring damage. On the other hand, there is a lot of controversy as to flood damage and you will find, after a major flood such as occurred in 1936, that you could go to Lawrence or Lowell and buy a house that had been flooded for half of what it was worth. It was what might be called a depressed value. They wanted to move out. Today you would pay more for those houses than you would in 1935, due to economic conditions. For that reason we do not take account of those historical losses in computing our damages.

CHLORINE DIOXIDE AS A BACTERICIDE IN WATER-WORKS OPERATION

BY R. N. ASTON AND JOHN F. SYNAN*

[*Read September 18, 1947.*]

WITH the increased use of chlorine dioxide for taste and odor control, there has been growing interest in the bactericidal properties of this gas. Investigation has proceeded along two lines: first, examination of the germicidal power of chlorine dioxide, when added to various samples of contaminated water in the laboratory; and secondly, study of the bacteriological improvement in waters which are being treated with chlorine dioxide for taste and odor control.

When the use of chlorine dioxide for removal of phenolic tastes and odors was first investigated in Niagara Falls in 1944, tests were also conducted on the bactericidal power of the gas. B. coli was added to distilled water and the efficiency of chlorine dioxide was compared with that of chlorine in the disinfection of the water. It was found that chlorine dioxide was more effective than chlorine as a bactericide for contaminated distilled water.

However, when water containing other organic contaminants was used, the picture changed somewhat. Chlorine dioxide owes its effectiveness as a destroyer of unpleasant tastes and odors to its high degree of reactivity with organic compounds. This same reactivity causes the chlorine dioxide to combine with phenols, algae and other organic pollutants at the same time as (or even before) it attacks bacteria. This is illustrated in Table 1, summarizing a number of such tests.

It may be seen from this table that, while the basic chlorine demand of these waters remains fairly constant, the chlorine-dioxide demand varies with the pollution. It is for this reason that the disinfection of certain polluted waters requires more chlorine dioxide than chlorine.

*The Mathieson Alkali Works, Inc., 60 E. 42d St., New York 17, N. Y.

TABLE 1.—RESULTS OF TESTS TO DETERMINE QUANTITIES OF CHLORINE DIOXIDE AND CHLORINE REQUIRED FOR DISINFECTION OF WATER.

Water	Coliform index of sample	Ppm for complete sterilization		Basic demand	
		ClO_2	Cl_2	ClO_2	Cl_2
(a) Merrimack River—tap water from Lawrence, Mass.—culture B. coli added	1,000,000	2.0	0.50		
(b) Merrimack River—untreated—filtered only	100,000	2.4	0.8-1.0	1.99	0.66
(c) Niagara River—tap water from Niagara Falls—culture B. coli added	1,000,000	0.6	0.6		
(d) Niagara River—untreated—filtered only	10,000	1.4	0.8	1.01	0.66
(e) Ohio River—untreated—filtered only—from Cincinnati	100,000	1.2	1.0	0.89	0.65
(f) Lake Michigan—untreated—filtered only—from Chicago—culture B. coli added	100,000	0.2	0.3	0.71	0.54

Similar conclusions have already been reached by Joseph A. McCarthy of the Massachusetts Department of Public Health, who conducted tests at the Lawrence Experiment Station. McCarthy, too, concluded that chlorine dioxide has definite bactericidal value but that much larger amounts are usually necessary than with chlorine. However, McCarthy pointed out several factors which have since proved to be important. In discussing the use of chlorine dioxide for taste and odor control, he suggested that the use of this gas could help to maintain a low bacterial count, through its destruction of organic material. In other words, by destroying organic material, the chlorine dioxide would permit the chlorine to act upon the bacteria without expending its energy on other organic matter. As will be indicated later, practical results in water plants using chlorine dioxide have borne out McCarthy's conclusion.

Still another point, brought out by McCarthy and since emphasized by other investigators, is the stability of the bactericidal action of chlorine dioxide with respect to pH. As McCarthy stated, "There is also a fair possibility that the bactericidal action may increase with increasing pH. Sufficient evidence is not yet at hand to give definite proof of this tendency, which is quite contrary to the action of chlorine, but, if it is confirmed, it may be of real value, especially since a high pH is now maintained in so many water supplies."

A further contribution to knowledge of the bactericidal action of chlorine dioxide was made by Ridenour and Ingols of the University of Michigan School of Public Health. Using the new ortho-tolidine-arsenite test for free residual chlorine, they compared the effectiveness of chlorine with that of chlorine dioxide in the inactivation of poliomyelitis virus. They found that the effectiveness of the free chlorine residual is independent of the type of chlorine-bearing compound used, the same results being obtained from chlorine, chlorine dioxide, or a mixture of the two, provided the material added was sufficient to give the required OTA residual. (Their tests show, by the way, that the amounts of free chlorine required to inactivate the virus were well within the range of practical dosages used in water treatment and swimming-pool sanitation.)

Following their investigation of chlorine dioxide as a source of free chlorine for disinfection, Ridenour and Ingols explored the bac-

tericidal properties of chlorine dioxide. A report on this work was published in the June, 1947, issue of the Journal of the American Water Works Association.

The experiments carried out by Ridenour and Ingols covered a variety of waters, including naturally polluted river water, artificially contaminated tap water, artificially polluted solutions of peptone in water, settled raw sewage, activated-sludge sewage effluent, and distilled water. These waters were buffered at pH 8 and tests were conducted at 75° C. The results are summarized in Table 2.

From the table it may be concluded, as the authors point out, that, regardless of the character or concentration of organic matter in the contaminated samples used for this study, chlorine dioxide was effective as a bactericide. Naturally, the amounts required for disinfection varied with the amount and nature of the organic matter in the sample. On the basis of weight, certain types of contamination require more chlorine and others more chlorine dioxide for kill, while some require about the same of each. Thus, when E. coli was added to Ann Arbor tap water, about the same amounts of chlorine dioxide or of chlorine were required for kill. As would be expected, raw sewage, which is high in organic content, required considerably more chlorine dioxide than chlorine. Mixtures of the two gases gave intermediate results.

The table also illustrates the already well-known fact that OT tests are much less accurate as indicators of free chlorine than the new OTA test. Thus, in all tests there was no consistent relationship between the OT residual and the bacterial kill. Based on the OTA test, however, there is a comparatively exact relationship between the residual and bacterial kill. We can see from the table that, with the exception of the first item, less than 0.15 ppm of OTA residual was sufficient with either chlorine or chlorine dioxide as the disinfectant. Based on the OTA residual, it may also be seen, chlorine dioxide is somewhat more effective as a disinfectant than chlorine. The average of all samples gave a required OTA residual of less than 0.06 ppm for chlorine dioxide, compared with 0.12 ppm for chlorine.

Ridenour and Ingols also tested the relationship between pH and disinfecting power, with results which bear out McCarthy's earlier suggestion. Waters at pH's adjusted to 6, 8 and 10 were used, with

TABLE 2.—BACTERICIDAL EFFECTS OF CHLORINE DIOXIDE, CHLORINE AND MIXTURE OF BOTH IN DIFFERENT MEDIA, ACCORDING TO RIDENOUR AND INGOLS.

		Chlorine dioxide			Chlorine			Mixture*		
			Residual at death point			Residual at death point			Residual at death point	
Type of pollution	Condition of sample	Weight applied (ppm)	(OT) ppm	(OTA) ppm	Weight applied (ppm)	(OT) ppm	(OTA) ppm	Weight applied (ppm)	(OT) ppm	(OTA) ppm
Natural	Huron R.	5.7	0.40	0.10	3.00	0.50	0.25	6.5	0.60	0.10
Esch. coli	Ann Arbor tap water	1.1	0.20	0.07	1.00	0.50	0.10	1.2	0.20	0.07
Esch. coli	200 ppm peptone	9.5	0.90	trace	12.00	3.30	0.12			
Esch. coli	400 ppm peptone	15.2	2.00	0.05	20.00	4.50	0.15	13.0	3.00	0.05
Natural	Activated-sludge effluent	1.9	0.40	0.10	1.00	0.20	0.10	1.5	0.25	0.10
Natural	Settled raw sewage	19.0	1.00	trace	10.00	trace	trace	20.3	0.15	trace
Esch. coli	Distilled water	0.8	0.10	0.05	1.00	0.50	0.10	1.1	0.20	0.07

*Chlorine to chlorine dioxide in 1:9 ratio.

TABLE 3.—SUMMARY OF EFFECT OF pH ON REQUIRED BACTERICIDAL CONCENTRATIONS OF CHLORINE DIOXIDE AND CHLORINE, ACCORDING TO RIDENOUR AND INGOLS.

		pH 6			pH 8			pH 10		
		Weight	Residual		Weight	Residual		Weight	Residual	
Substrate	Disinfectant	applied (ppm)	(OT) ppm	(OTA) ppm	applied (ppm)	(OT) ppm	(OTA) ppm	applied (ppm)	(OT) ppm	(OTA) ppm
River water	ClO_2	1.0	0.10	0	0.6	0.10	0	0.8	0.1	0
River water	Cl	1.0	0.15	trace	2.0	0.25	0.05	3.0	0.6	0.10
Peptone solution*	ClO_2	2.3	0.05	0	2.9	0.40	0.05	3.0	0.3	0
Peptone solution*	Cl	2.5	1.50	0.08	2.5	1.20	0.05	8.0	3.5	0.15

*50 ppm.

results summarized in Table 3. Thus, when the pH of river water was raised from 6 to 10, the weight of chlorine required for disinfection rose from 1 ppm to 3.0 ppm. By contrast, the weight of chlorine dioxide required actually fell from 1 ppm to 0.8 ppm. Similarly, when peptone solution is disinfected, the chlorine required is multiplied by more than 3 as the pH rises from 6 to 10, whereas there is only a slight rise in the amount of chlorine dioxide required.

This stability of the disinfecting power of chlorine dioxide with rising pH, as the authors point out, indicates "important applications in waters of high natural alkalinity, those treated to prevent corrosion and softened waters."

These laboratory tests lead us to expect that waters treated with chlorine dioxide should show an improved bacterial quality. There are practical considerations which lead us to the same conclusion. It is well known, for example, that many water plants have had difficulty in maintaining a residual to the end of the distribution lines. This difficulty comes from the fact that organic materials, which react only slowly with chlorine, can come through the filters and then consume the residual chlorine in the distribution system. As stated earlier, chlorine dioxide reacts rapidly with these organic compounds, thus making it possible to maintain an adequate residual to the end of the lines.

Similarly, many communities cannot completely disinfect by prechlorination, because of the severe tastes and odors developed. Application of chlorine dioxide, however, makes adequate pre-chlorination possible, thus permitting effective disinfection before the water is filtered.

The bactericidal properties of the chlorine dioxide itself may result in a further improvement in the bacterial quality of the effluent water. As an example of this effect we may cite the experience at Chicopee, Mass.

Chlorine-dioxide treatment was initiated in January, 1945, at the Chicopee water plant, to overcome a severe taste and odor problem. Prechlorination had been impossible, because of the intense taste and odor developed. Post-chlorination increased the taste and odor to a lesser degree, but the balance between the amount of chlorine for disinfection and the amount that made the taste and odor of the

TABLE 4.—MONTHLY AVERAGES OF BACTERIOLOGICAL ANALYSES OF CHICOPEE WATER.

	1944				1945			
	Raw water		Final effluent		Raw water		Final effluent	
	Bacteria per cc		Bacteria per cc		Bacteria per cc		Bacteria per cc	
	2 days, 20° C. N. agar	Coli-aerogenes* % pos.	2 days, 20° C. N. agar	Coli-aerogenes* % pos.	2 days, 20° C. N. agar	Coli-aerogenes* % pos.	2 days, 20° C. N. agar	Coli-aerogenes* % pos.
Jan.	9,000	12	13	0	55,000	75	6	0
Feb.	25,000	33	18	0	29,000	25	1	0
Mar.	9,000	88	140	1	18,000	25	7	0
Apr.	3,600	17	47	0	16,000	100	1	0
May	1,000	50	10	0	46,000	100	6	0
June	14,000	83	40	2	13,000	100	5	0
July	3,800	100	31	0	6,000	100	3	0
Aug.	14,000	100	40	7	1,700	83	5	0
Sept.	47,000	100	17	0	1,500	50	3	0
Oct.	14,000	100	3	0	2,300	100	1	0
Nov.	22,000	50	22	0	2,300	83	1	0
Dec.	25,000	75	10	0	3,200	17	1	0

*In 10 cc lactose broth.

finished water exceedingly unpalatable was so fine that the potability of the finished water was always close to the danger point.

The introduction of chlorine-dioxide treatment had the expected results. It was possible to pre-chlorinate for disinfection, without regard to taste and odor difficulties. Post-treatment with chlorine dioxide eliminated tastes and odors and, in addition, made it possible to maintain a residual to the end of the line. A description of the installation is given in the McCarthy report mentioned earlier. Here we are concerned chiefly with the effect of chlorine-dioxide treatment on the bacterial content of the water. This is summarized in Table 4.

Here we have listed the monthly averages of bacterial analyses for 1944, the year before the installation of chlorine-dioxide treatment, and for 1945, the first year of chlorine-dioxide treatment. Figures for raw water and final effluent are given. It can be seen from this table that, although the raw water was about the same for both years, there was a marked improvement in the bacterial content of the water in 1945. For example, the average number of bacteria per cc in the final effluent was considerably higher in 1944 than in 1945. Moreover, since the introduction of chlorine-dioxide treatment, tests for coliforms have been consistently negative.

There has also been a consistent improvement in the bacterial quality of the water since 1945, as may be seen from Table 5, which

TABLE 5.—BACTERIOLOGICAL ANALYSES OF FINAL EFFLUENT AT CHICOPEE, MASS., BEFORE AND AFTER CHLORINE-DIOXIDE TREATMENT.

	Before		After		
	1943	1944	1945	1946	1947*
2 days, 20° C.—Nutrient Agar					
Bacteria per cc: Average	45	41	3.3	1	—
Maximum	210	140	7	2	2
24 hours, 37° C.—Purple Agar					
Bacteria per cc: Average	2.3	3	0.7	0.3	—
Maximum	10	7	7	1	1
Months Negative	0	0	5	8	—
Coli-Aerogenes in 10ml Lactose Broth					
% Positive, Maximum	5	7	0	0	0
Months Negative	7	9	12	12	12

*First 6 months.

gives the figures for the two years preceding chlorine-dioxide treatment and the 2½ years since that time. The test conducted on nutrient agar for two days at 20° C. gave an average bacteria count of 45 per cc in 1943 and 41 in 1944. In 1945 the average dropped to 3.3 and in 1946 the average was 1.0. The maximum figure for this same test was 210 in 1943 and 140 in 1944. In 1945 it had dropped to 7, and in 1946 and the first six months of 1947 the maximum was further reduced to 2.

A similar, marked and continuously growing improvement is shown by the bacteriological test conducted on purple agar for 24 hours at 37° C. Here the maximum dropped from 7 to 1 bacteria per cc after the introduction of chlorine-dioxide treatment. The consistently negative results for coliform organisms continued through the first six months of this year.

On the basis of our present knowledge, therefore, we may conclude that chlorine-dioxide application leads to an improvement in the bacterial quality of water, as a result of one or more of the following factors:

1. Chlorine-dioxide water treatment often permits effective disinfection by pre-chlorination, where it would otherwise be impossible.
2. Chlorine-dioxide treatment makes it possible to maintain a residual to the end of the distribution lines.
3. Chlorine dioxide is an effective bactericide, in all types of contaminated water, and is even more efficient than chlorine, where organic contamination is low.
4. Chlorine-dioxide bactericidal action is stable with respect to pH and thus makes chlorine-dioxide treatment especially valuable in waters where the pH is high.

Discussion

Joseph A. McCarthy (Lawrence, Mass.). I was particularly interested in following the bacteriological counts in Chicopee, at what was the most convenient and at the same time most distant point on the system from the pumping station. I should say now, from memory, that, about three weeks after the chlorine-dioxide treatment was stabilized, they began to notice a drop in the 20° counts. They began to drop and stayed down. The last samples that

I have seen from that point had practically the same 20° or 37° count at the extreme end of the system as at the pumping station. It was my guess, in connection with the earliest paper on this subject, that chlorine dioxide would be a very valuable vehicle for chlorine and it seems that my guess at this distance was pretty good.

WARREN J. SCOTT (Hartford, Conn.). I would like to ask how many are using this chlorine dioxide in the way described.

R. N. ASTON (New York, N. Y.). I would say that it is in operation now in about 65 cities in this country and about 12 or 15 in Canada. They are scattered from New England to Florida and west as far as Minnesota, and also in California. Hence it is used on a variety of types of water.

MR. SCOTT. How many cities use it where they have supplies which are not filtered, and are those ones that have objectionable tastes in the water?

MR. ASTON. Yes, in the case of two in Michigan. In the case of Monroe, where it is an algae problem, they get fishy and musty tastes.

ARTHUR L. SHAW (Boston, Mass.) Mr. Aston has very carefully refrained from even inferring that he works for a company which sells both chlorine and chlorine dioxide. I think there may be some of us here who have not the brains to understand the chemistry and biology of this thing, and such people may be wondering what the chemical significance of all these chemicals is. Is it advantageous to your company and other companies who make these chemicals to sell one in preference to the other?

MR. ASTON. Why, I would say the answer as to which chemical should be sold is—I should add to that, there is no city, where chlorine dioxide is installed and being used, where it is being used primarily as a disinfecting agent. For example, in Chicopee it is used with chlorine. Now, until there is sufficient bacteriological evidence, both in the laboratory and at the plant, there is no one who would want to try to use it primarily as a disinfecting agent, but we are trying to gather the evidence to prove it and then we can go out and offer it as the sole means of disinfecting.

In regard to what we prefer to sell—chlorine or chlorine dioxide—I would say that the answer to that is: whichever would do the best job. It would be foolish for anybody to sell a product because the

profit would be higher. That would not be a sound basis. So that I would say the answer is to use whichever does the job—whichever will stay there and produce the results. Chlorine, of course, is used in large quantities, so that you get into the cost, in one instance, of using in a big pond 50 or 60 lb of chlorine as against 2 or 3 lb, which is the average dose of sodium chlorite being used throughout the country. While one is a more expensive chemical, the amount sold is much less.

MR. SHAW. I was interested to know whether you had a by-product, for which your research department was seeking an outlet.

MR. ASTON. No, definitely not that.

MR. MCCARTHY. There is another little interesting product of chlorine dioxide. I think I used it in a very small sample of water—the ordinary sample. By using a mixture of chlorine and chlorine dioxide I was able to reduce the taste very sharply, to eliminate most of the manganese and the musty or iron taste, and to kill the bacteria, at a pretty stiff cost.

GEORGE B. GARRETT, JR. The high-pH chloramines were much more effective than the chlorines and today the chlorine dioxide is more effective than the chlorine. The discrepancy is due to the formation of hypochlorous acid. With these facts you point out that that is not the true chemical nature of the disinfectant. Can you enlighten us on that?

MR. ASTON. In the case of chlorine dioxide you do not have hypochlorous acid, so that, in considering chlorine, chloramine and chlorine dioxide, you do not have a common agent. It is hypochlorous acid which you have in chlorine and chloramine. The only similarity would be that they are all oxidative reactions. That is going to the fundamental subject of bacterial action and I am afraid I can't offer anything on that. There are others much more capable of answering that.

If I may, I would add to Mr. McCarthy's story on manganese. I have about three or four plants now using chlorine dioxide for manganese and iron removal. In the one plant where we have obtained data, their average dose is running about 2 lb per mil gal with sodium chlorite. Incidentally, we have found, in trying to do laboratory work on manganese removal, that the laboratory work indicated

we would need about 10 lb per mil gal but, on going into the plant, we found we could operate at about 2 lb.

A MEMBER. I would like to ask Mr. Aston whether he would care to comment on the possibility of using some convenient form of tablet. Is it possible to get chlorine dioxide in such form now or do you think there is any possibility of developing something on that?

MR. ASTON. The nearest I could come to answering that is to say that you do not have to use chlorine dioxide but perhaps an acid. We might be able, at some time, to produce a tablet. What you are getting at is some convenient unit package, which will represent the chlorine dioxide that we have today. I think it may be possible in the future.

MORTIMER M. GIBBONS (Rahway, N. J.). I would like to say that we use chlorine dioxide in the municipal water supply at Rahway, N. J., now.

I am happy to say that many of the things Mr. Aston has claimed for his product are true, but he did mention manganese. It appears to us that there are some disadvantages in the way of physical deterioration. For instance, if you have manganese coming through in your filter effluent and add chlorine dioxide to the filtered water at the entrance to the pure well, the manganese will be precipitated, and that may discolor the water or result in a dirtier-appearing water. In addition to that, there is apparently some increase in the color of the filtered water, due possibly to some other substance which is in the water—possibly tannin. In that case the color can increase very decidedly, as you increase your dose of chlorine dioxide. I mention that to indicate that there are some practical difficulties in applying this to a municipal water supply—it would have to be investigated—although I believe the advantages outweigh the disadvantages.

MR. ASTON. I would add one point to that. Now we are removing the manganese, as I hinted, in all those instances, as we now apply the chlorine dioxide to the raw water used in the mixing chamber, so that the manganese is oxidized and settled out in the settling basin before filtration.

GILBERT H. PRATT (Gloucester, Mass.). If I understand rightly, you use chlorine in the manufacture of chlorine dioxide, do you not?

MR. ASTON. That is correct.

MR. PRATT. That is the only way it is produced at the present time, isn't it?

MR. ASTON. That is right.

MR. PRATT. How much chlorine do you have to use? You spoke of the 2-pound basis.

MR. ASTON. The theoretical amount is about 1 lb of chlorine and 3 lb of chlorite, but we recommend about 1 lb of chlorine to 2 lb of chlorite for generation of chlorine dioxide.

MARSHALL S. WELLINGTON (New Haven, Conn.). In reading the literature about this, hearing about the people who have tried it, and hearing the statements that have been made here today, it seems to me that, when they use this chlorine dioxide, they have always increased the chlorine before they used it. I would like to know what the reason for increasing the dose is.

MR. ASTON. In some instances we have increased the amount of chlorination but that is not universally so. Particularly in an algae problem, we find that, if we chlorinate more heavily initially and then react with chlorine dioxide, we get a better taste and odor removal than otherwise. As to why, we would like to know ourselves, but I believe it is because we perhaps form chlorinated compounds, which react more readily.

One point, which I have mentioned rather vaguely, is that chlorine dioxide does have many specific reactions, and that is illustrated by the fact that chlorine dioxide, for example, will not react with ammonia in the water or with chloramine. That is an advantage in many instances but may prove to be a disadvantage when chlorine dioxide has specific reactions, because we may well find that there are certain types of tastes and odors that we can't remove, because chlorine dioxide won't react.

MR. WELLINGTON. Don't you think that perhaps increase in dose would reduce your bacterial count?

MR. ASTON. Yes, it probably would in that case.

MR. WELLINGTON. So that you could not say that the chlorine dioxide had the advantage of reducing your counts?

MR. ASTON. I think I do, until we can get more evidence on it.

LINN H. ENSLOW (New York, N. Y). Would you put chlorine

dioxide in alone for iron removal or do you require some chlorine for that also?

MR. ASTON. We are not operating without chlorine, because we are not removing any point of chlorination, because we do not want to interfere in any way with the disinfection of the water. We could do it without chlorine at all—I think we could—but, practically speaking, we would not, because that would then give us only one point of chlorination. We would only have, in the town we are operating in, a chance to chlorinate the finished water and we would be depending much more on the chlorine dioxide for disinfectant.

A MEMBER. Isn't there an economic reason for using chlorine rather than chlorine dioxide, as one costs about 7½ cents and the other 75 cents a pound?

MR. ASTON. Yes. Economically you could disinfect much cheaper with chlorine than with chlorine dioxide.

A MEMBER. Nothing has been said about the effect of this type of treatment on the organisms other than the coliform organisms. Has anything been done along that line?

MR. ASTON. I would assume you mean what is called the presumptive-type organisms. There has been definite evidence of the reduction of the presumptives. We are now supporting some studies at the University of Michigan, where we are going to investigate those particular organisms, to see if, on those, there may be again a specific reaction with chlorine dioxide, but there has been evidence in the plants of reduction of presumptives.

BEHAVIOR OF OVERLOADED FILTERS

BY L. F. DUTTON**

[*Received January 8, 1948.*]

A BRIEF chronicle of experience with rapid sand filters, which have been continuously overloaded for several years, as a result of war-related increases in demand and war-time restriction on expansion, will add to the information available on the overload capacity of such units. From time to time question is raised as to the suitability of the conventional filtration rates customarily adopted for design and as to the wisdom of increasing them with a view to reduction of first cost. It has been demonstrated at the plant in question that rates substantially higher than normal can be handled for long periods without undue physical difficulty.

The water-treatment plant at Groton, Conn.,* has a nominal filter capacity of 2 mgd on the basis of the conventional rate of 125 mgda (million gallons per day per acre) or 2 gpm per sq ft. The filters were operated at this rate or less for about three years, until the last of 1942. For the following four years the filter rates were continuously above normal, averaging over 170 mgda (36% overload) for the two years, 1944 and 1945, and recording a maximum month of 186 mgda (49% overload), with other consecutive months close to this maximum. The filter rates are plotted on Figure 1.

As an aid to the detection of physical deterioration in the filters, the monthly operating records have been used to determine the average output of the filter units in each cycle between washings. These quantities have been plotted by months and as annual averages on Figure 1 in terms of gallons per cycle, per filter unit of 175.2 sq ft. If the filters were degenerating from continuous overload, some evidence of it would presumably appear in progressive lessening of the quantity passed per cycle.

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*This JOURNAL, Vol. LV, No. 2, p. 233, 1941.

**Superintendent, Dept. of Utilities, Groton, Conn.

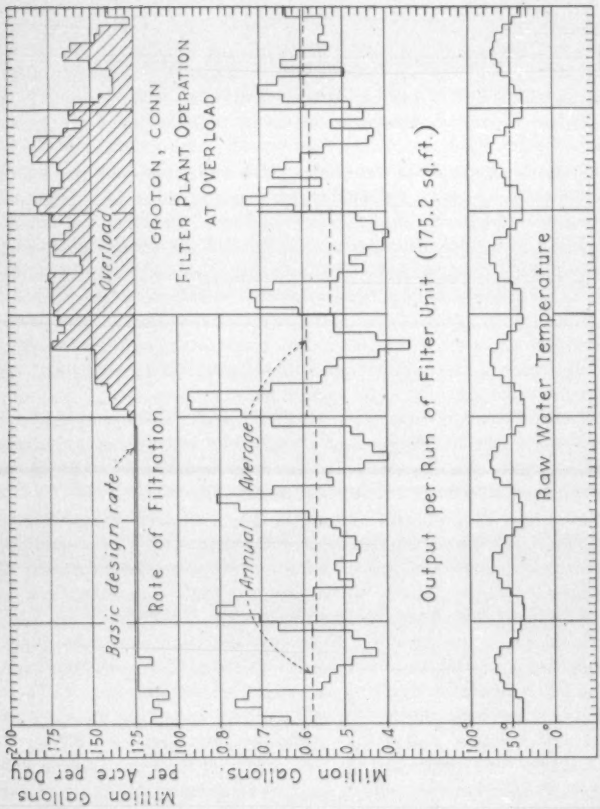
Million Gallons per Acre per Day
200
175
150
125
100
Basic design rate
Overload
Rate of Filtration
GROTON, CONN
FILTER PLANT OPERATION
AT OVERLOAD
Million Gallons
0.8
0.7
0.6
0.5
0.4
Annual Average
Output per Run of Filter Unit (175.2 sq.ft.)
100°
50°
0°
Raw Water Temperature
1940
1941
1942
1943
1944
1945
1946

FIG. 1.

Reduction in length of filter runs is to be expected with higher rates of filtration but, if the quantity passed per cycle remains relatively undiminished, the filters may be considered to be maintaining their effectiveness, other things being equal. It will be seen that the annual average quantities per cycle are reasonably uniform from year to year and give no evidence of significant filter deterioration.

A marked seasonal pattern of variation in unit output is noticeable, revealing an interesting relation between quantity filtered and temperature of raw water. In general, the quantities per cycle are higher, the lower the temperature. This is contrary to theoretical considerations related to the higher viscosity of cold water but is probably accounted for by the greater prevalence of algae and other organisms in warmer water, with resultant shortening of filter runs.

Percentage of wash water, consistently remaining within limits of 2.5% to 4.5%, gives no indication of distress in the condition of the filters, which have continued to produce a satisfactory effluent, having a color, with rare exceptions, of 5 or less.

Most of the operating problems encountered could be traced to preliminary treatment and resulted from the shortening of detention periods for mixing and settling at rates of treatment in excess of the design. Some inconvenience was experienced, due to the inability to put filter units out of service long enough for such routine work as painting wash-water troughs, and the like. Routine hand-raking of the beds during washing has been an important factor in preventing the formation of compacted areas, which might have resulted in progressive loss of effectiveness. Regular hand-raking has proven to be not unduly laborious on these relatively small filter units. Much credit is due the operators at the Groton plant for the successful way in which continuously overloaded conditions have been met. The periodic review of operating results and comments and suggestions by the engineers of the State Health Department, and by Metcalf & Eddy, who designed the plant, have also been helpful.

It may be concluded from the experience at this plant that, with careful, intelligent operation, it is possible to operate filters for long periods at rates approaching 3 gpm per sq ft. However, filters to be operated at this rate should be preceded by preliminary treatment facilities of commensurate size and some reserve filter capacity should

be provided, to permit ordinary maintenance work. Filters designed for the higher rate would have little or no further capacity for overload. If the plant in question had been designed with filter area based on 3 gpm per sq ft, it is very doubtful that it could have been successfully accelerated to as high a rate as would have been necessary to meet the demands upon it.

Notwithstanding the ability of filters to operate at high rates, it would appear to be sound practice to adopt conservative basic-design rates and to reserve the known ability to operate at overload as a factor of safety against unforeseen emergencies. It is, of course, necessary that all such details as pipe sizes, rate-controller capacities and available head be in keeping with the higher rate, if the maximum capacity of the filters is to be fully available, when the need arises.

PROCEEDINGS

November 1947 Meeting

Hotel Statler, Boston, Mass.

Thursday, November 20, 1947

President Donald C. Calderwood in the Chair.

Secretary Knox announced the election to membership of Edward J. Kelley, Business Agent, Construction Division, Metropolitan District Commission, Boston, Mass.

A paper on "Work of the New England Forestry Foundation on Watersheds" was read by Harris A. Reynolds, Secretary, New England Forestry Foundation, Boston, Mass.

A paper on "Small Ground-Water Supply Systems for State Parks" was read by Arnold E. Howard, Park Engineer, Division of Parks and Recreation, Massachusetts Department of Conservation, Boston, Mass.

H. B. Kinnison, District Engineer, U. S. Geological Survey, Boston, Mass., gave a talk on "The Current Drought in New England."

December 1947 Meeting

Hotel Statler, Boston, Mass.

Thursday, December 18, 1947

President Donald C. Calderwood in the Chair.

Secretary Knox announced the election of the following new members:

Members: John S. Capaldi, Vice-President, Capaldi Bros. Construction Co., Providence, R. I.; R. Nelson Molt, Administrative Office Manager, Metropolitan District Commission, Construction Division, Boston, Mass.; Walter L. Olson, Water Foreman, City of Melrose, Mass.; Richard O'Neill, Water Foreman, City of Peabody,

Mass.; William A. D. Wurts, District Manager, Metropolitan District, Hartford, Conn.

Associate: Bowser, Inc., Manufacturers of Water Filters, Fort Wayne, Ind.

A paper on "Reflections on the Bar Harbor Fire" was read by Theodore W. Grindle, Superintendent, Bar Harbor Water Co., Bar Harbor, Me.

A paper on "Forest Fire Hazards" was read by Raymond E. Rendall, State Forest Commissioner, Augusta, Me.

A paper on "Experiences during the Recent Fires in York County, Maine," was read by L. G. Smith, Superintendent and Treasurer, Kennebunk Water District, Kennebunk, Me.

JANUARY 1948 MEETING

HOTEL STATLER, BOSTON, MASS.

Thursday, January 15, 1948

President Donald C. Calderwood in the Chair.

Secretary Knox announced the election of the following new members: William H. Balke, Jr., Division Engineer, Connecticut Light & Power Co., Enfield, Conn.; J. Elliott Hale, Superintendent, Kennebec Water District, Waterville, Me.; Harry W. Lea, Consulting Engineer, Montreal, Quebec, Canada; Cecil W. Mann, Superintendent, Livermore Falls Water and Sewer District, Livermore Falls, Me.

President Calderwood read the following statement by the Executive Committee:

"In accordance with the vote of the Association on September 16, 1947, the Executive Committee has reconsidered its previous action relative to the publication of the paper entitled 'Contractors' Problems in Construction of Water Distribution Systems', and it was voted that it is for the best interests of the Association not to publish the paper."

Arthur C. King, Chairman of the Committee on Legislation, presented the following report: Mr. President and members: Your

Committee on Legislation, with the approval of the Executive Board, prepared a rather short and simple bill and presented it to the Massachusetts Legislature at this session. Without much warning there was a hearing on it day before yesterday, before the Committee on Water Supply. The committee, I believe, was quite favorable but it was decided, after talking it over in a friendly way, that probably the best procedure would be for that Committee on Water Supply to bring in a resolve to have a recess commission make a thorough study of not only this bill but all of the allied subjects, bringing in the Department of Accounts, as well as the Department of Public Health and also the mayors and selectmen, and report probably at the next session. That will probably be the action of the committee but I will tell you just in a few words what the bill covers.

It is Senate bill 417, entitled "An Act Relative to the Development and Use by Cities and Towns of Water Systems and Sources of Water Supply." I won't read it but I will give you the idea, which is that the Water Commissioners shall have full charge, control and management of a public system, that all water-department revenue shall be held for water-works purposes, that a depreciation fund shall be set up and held for that sort of thing, that the municipalities shall pay their water departments annual charges for fire protection, as well as for other municipal uses, and that, if desired by the city or town—this last paragraph is only optional—the water department may pay to the municipality something in lieu of taxes, to be based on plant value, but this sum in lieu of taxes shall not exceed 3% of the plant value. That is the gist of the bill as presented. I thank you.

A paper on "Gypsy Moth Control with DDT" was read by John M. Corliss, in Charge of the Division of Gypsy and Brown-tail Moths Control, United States Department of Agriculture, Greenfield, Mass.

A paper on "The Algal Environment in Relation to Fish" was read by Allan S. Kennedy, Assistant Fish and Game Biologist, Division of Fisheries and Game, Massachusetts Department of Conservation, Boston, Mass.

A talk on "Financing Public Works in New England" was given by John W. Agnew, Manager, Municipal Division, First National Bank, Boston, Mass.

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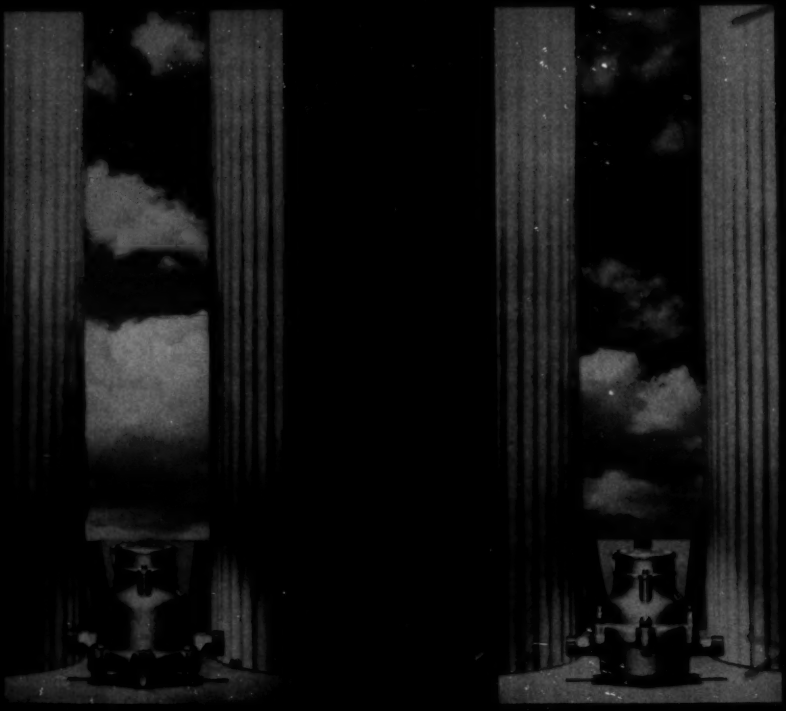

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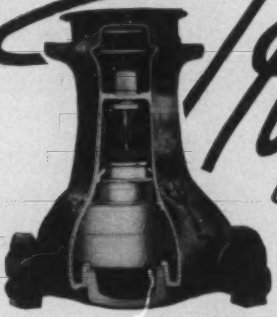

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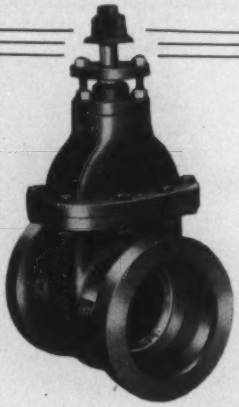

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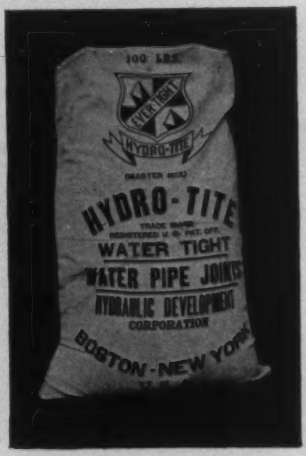

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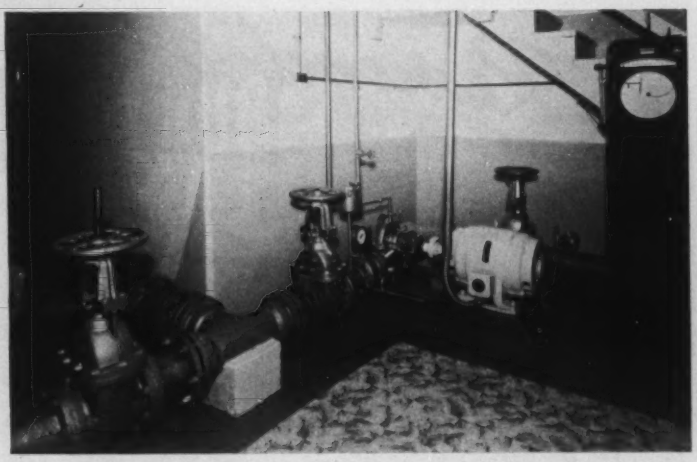

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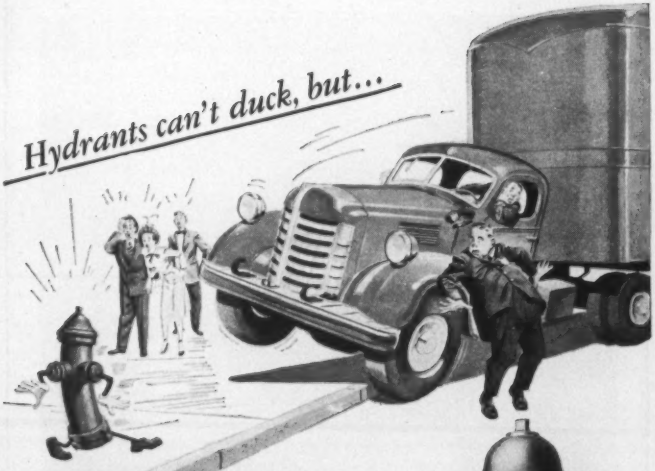

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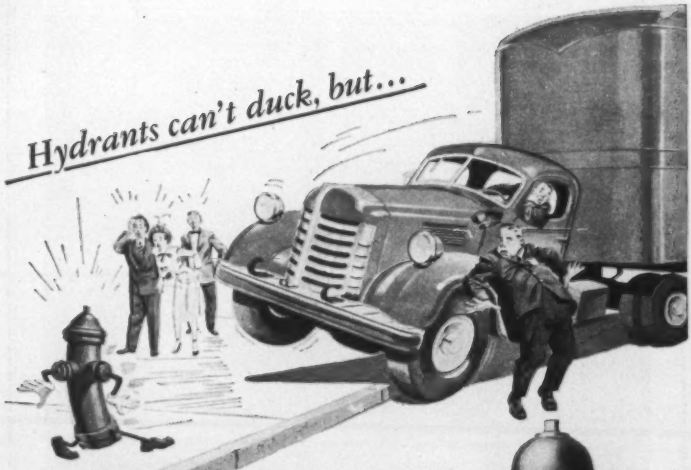


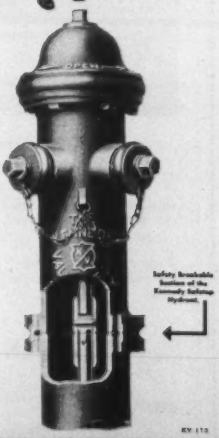
Kennedy Safetops
do the next best thing

A fire hydrant hasn't been made that can dodge an accident. But when a Kennedy Safetop Hydrant is struck a smashing blow which no hydrant can withstand, only the Safety Breakable Section gives way. The essential working parts of the hydrant are left undamaged, the danger of injury to persons or property is minimized and the cost of repair is far less.

There is no flood or geyser . . . no need to shut off local water supply . . . no need for excavation or complete replacement. One man, with but $10 worth of parts, can put the hydrant back in service in half an hour.

For extra safety and extra savings . . . standardize on Safetops.

THE KENNEDY VALVE MFG. CO. • ELMIRA, NEW YORK

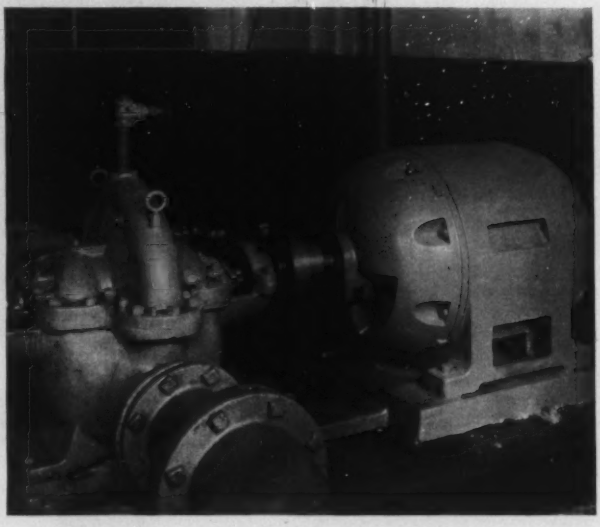


KV-175

KENNEDY Safetop HYDRANT

DE LAVAL

CENTRIFUGAL WATER WORKS PUMPS



Town of Clinton, Mass., Water Works

A recent installation of a modern heavy duty De Laval water works pump designed for 1500 G.P.M., against 232 feet net head with 65 feet positive head on the suction. Special sleeve construction is employed because of the high positive suction pressure.

This new unit replaces an old De Laval pump that was retired after more than 25 years of satisfactory service because of increased water demand.

TURBINE EQUIPMENT COMPANY OF NEW ENGLAND

80 Federal Street **Phone LIBerty 5993-5994** **Boston, Mass.**

New England Representatives for

DE LAVAL STEAM TURBINE COMPANY

TRENTON, NEW JERSEY

Ingot or Wool?

Whether the caulking job calls for ingots for cast lead joints or lead wool for cold caulking, the primary requirement in either case is lead of high purity.

Don't take chances with inferior caulking lead which may have been manufactured from scrap material containing a large percentage of impurities.

When present in a caulked joint, impurities on the surface will corrode and cause weak spots. The stresses due to normal pipe movement concentrate at these areas and failure results.

Our lead, both in ingot and wool form, is made from selected metals and carries the Lead Industries Association Seal of Approval as a guarantee of its quality.

OTHER PRODUCTS—
LEAD PIPE
SOLDER
WHITE LEAD
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Within the Boston Edison Company is a staff of experienced men thoroughly qualified to give you advice on any branch of our electric service.

Whether you are interested in better lighting — air conditioning — industrial heating — commercial cooking or any other electrical improvement the assistance of these men is available to Boston Edison customers without cost or obligation.

Call or write to the

Electric and Steam Sales Department
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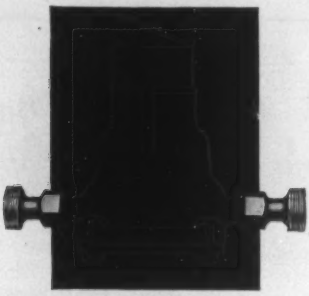
BOSTON EDISON COMPANY



WHEN WATER LINES ARE

OVER-METERED

You can put correct metering on over-sized lines with Ford Meter Adaptors. These handy, inexpensive connectors permit use of smaller meters for better registration of smaller flows. Just screw them onto the smaller meters and install where larger meters have been. You get the benefit of more accurate water measure.

FREE CATALOG

Send for new Ford catalog showing Adaptors and many other money-saving meter setting devices. Send today.

FORD METER BOX CO.
WABASH, INDIANA

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METER ADAPTORS

THE HEFFERNAN PRESS

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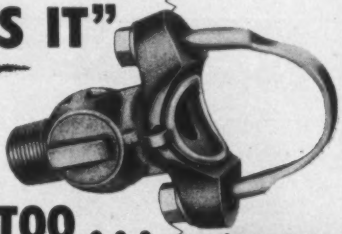
150 FREMONT STREET

WORCESTER, MASSACHUSETTS

"EASY DOES IT"



QUICKER— STRONGER, TOO . . .

Patent Applied For

with HAYS "DUO-STOP"

Combination Corporation Stop and Saddle for Cast Iron or Steel Pipe

It's quick . . . it's simple . . . it's safe. It's the corporation stop that water companies and municipalities have needed for a long time. Especially today, with the large scale housing projects and the demand they put on the service department, the Hays "Duo-Stop" is an important time-saver in making connections to small mains.

There's no interruption of service . . . there's no tapping of the main . . . there's no weak spot in the connection.

The "Duo-Stop" Combination Corporation Stop and Saddle is strong . . . stop and service clamp are of one-piece solid, rust-proof bronze . . . it is pressure-tight and leakless.

The Hays "Duo-Stop" is built to serve and built to last.

Write today for Folder 105

HAYS

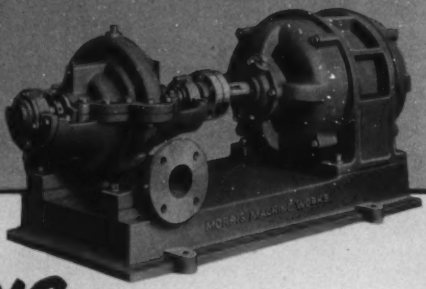
COPPER · BRASS · LEAD · IRON

WATER WORKS PRODUCTS

HAYS MANUFACTURING CO., ERIE, PA.

FLOATING Sealing Rings...

Permit CLOSER Running Clearances



MORRIS HORIZONTAL SPLIT Double-Suction WATER PUMP

No other pump has the Floating Sealing Rings . . . yet this exclusive feature alone makes the Morris Double-Suction Water Pump outstanding for efficiency and long service.

During the instant that the pump comes up to speed, but before full pressure is built up, the Morris Floating Sealing Rings adjust themselves to a uniformly close running clearance around the impeller. As soon as full speed is developed, the internal hydraulic pressure acting upon the vertical face of the Floating Ring, presses it tightly against the precision machined surface of the casing ring where it is firmly held in place.

This action permits machining the rings to closer clearances. Leakage is consequently reduced to a minimum, initial efficiency maintained for much longer periods of service. Assures concentricity of ring with impeller and eliminates binding due to abnormal operating conditions.

Advanced design of Morris Double-Suction Water Pumps and their several exclusive features assure increased hydraulic and mechanical efficiency . . . reliability . . . and low maintenance cost. Write today for free Bulletin 179.

MORRIS MACHINE WORKS
Baldwinsville, N. Y.
Sales Offices in Principal Cities

MORRIS *Centrifugal Pumps*

EDDY

Valves and Hydrants

Distinctive Features of the EDDY Valves

ONLY three parts are moved by the Stem—the ball and the two gates. Each gate is hung from one of the trunnions on the ball. A convex surface at the base of each trunnion fits a concave surface on the back of the gate. This allows the gates to adjust themselves properly to their seats. . . . The gates, being free to revolve on the trunnions, do not always seat in the same position. Gates are center bearing and adjustable. They are forced to their seats with equal pressure at all points. There are two hooks on the ball on sizes 4-inch and larger, which loosely engage with the gates.

Distinctive Features of Our New Swivel Top Hydrants

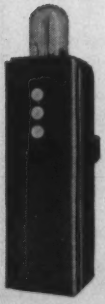
THIS new hydrant has all the advantages of the popular EDDY fire hydrant, plus several additional features. . . . Nozzles are in a short flanged section of the standpipe, which may be unbolted and turned to different positions. . . . Should the standpipe be broken, only the casting below the swivel head need be replaced. . . . To raise the hydrant to conform to a new grading, simply insert a flanged extension piece below the swivelhead. No digging. To add a steamer nozzle at any time, it is only necessary to replace the swivel head with one having a steamer nozzle.

Besides valves and hydrants to meet the most exacting demands of waterworks service, the complete EDDY Line includes: valves for sewage disposal works; valves for steam, gas and oil; check valves, foot valves, plug valves, shear gates, indicator posts, etc. EDDY valves are made in three classes: Iron Body Bronze Mounted, All Bronze, and All Iron Valves.

Eddy Valve Co.

WATERFORD NEW YORK

'Break Point' Chlorination Gives Hagerstown Much 'Sweeter' Water
That unpleasant "after" taste in your drinking water is gone and whether you believe it or not it is an extra hearty "shot" of chlorine daily that is giving Hagerstown consumers the purest, sweetest, flavored mountain water they have ever had.
And it's only costing the taxpayers around $1.50 more daily to get the improved water.
The "break point" chlorination principle, now in use in the water systems of approximately 70 American cities, including both Baltimore and Washington, was first introduced into two weeks mountain water about two weeks ago. Extensive laboratory studies and tests were made by Robert W. King, chemist for the Water Department, before the additional chlorine was added to the treatment of water coming from the city's two mountain reservoirs.
Under the "break point" principle, enough chlorine is added to the water to burn up all organic matter, plant and bacteria. Heretofore, ammonia also was used along with chlorine in purifying the water, but a six year study by leaders in the chlorine industry showed that when ammonia is used along with chlorine the after taste is accentuated. By use of additional chlorine and no ammonia it has been found that there is no after taste once the amount of necessary chlorine to completely burn up plant matter and bacteria is ascertained.
Tests made by the local chemist showed that about 50 percent more chlorine than was used with ammonia was needed to readjust the water 100 percent free of bacteria and at the same time leaving no after taste.
Now studies are being made of the water from the Potomac river, which makes up about one-third of the water supplied to Hagerstown daily, with an idea later of using the "break point" principle on the river water.
Several years ago the Potomac river water's flavor also was considerably improved by the use of black alum. It was not uncommon during the summer months, when this "river" flavor was most noticeable, for the water department office to be swamped with complaints on the "taste" of the water. But the use of black alum removed this disagreeable flavor and the complaints have been few.
Reproduced from the Hagerstown, Md. Daily Mail, March 29, 1946



This is but one of the many benefits of the Break-Point Process. Your W & T Representative will be glad to tell you how it can also give you improved coagulation, longer filter runs, better algae control, help in iron, manganese and color removal and the elimination of all gas-formers.

"THE ONLY SAFE WATER IS A STERILIZED WATER"

WALLACE & TIERNAN
COMPANY, INC.

MANUFACTURERS OF CHLORINE AND AMMONIA CONTROL APPARATUS
NEWARK 1, NEW JERSEY • REPRESENTED IN PRINCIPAL CITIES

S-14

EDSON

PUMPS and ACCESSORIES

Hand Pumps - Power Pumps
Truck or Trailer Mounted
Edson Special Suction Hose
Red Seal Diaphragms
Edson Bronze Hydrant Pump
Strainers, Adapters, etc.

Distributors for
Pollard Pipe Line Equipment
Electric Pipe Thawers
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New York: 142 Ashland Place, Brooklyn

Warren Foundry & Pipe Corp.

ALSO

Warren Pipe Co. of Mass., Inc.

SALES OFFICES
11 BROADWAY, NEW YORK
75 FEDERAL STREET, BOSTON, MASS.

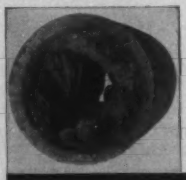
Manufacturers of

CAST IRON PIPE

Flanged Pipe *Flexible Joint Pipe*
Bell and Spigot Pipe
Special Castings *Short Body B. & S. Specials*
Warren (WF) Spun Centrifugally Cast Iron Pipe

WORKS: PHILLIPSBURG, N. J. and EVERETT, MASS.

Large Stock Enables Us to Make Prompt Shipments



Get Full Service From Old Mains

The most practical way to meet today's overload demand is to clean clogged water mains. FLEXIBLE Underground Pipe Cleaning Co., specializes in such jobs— any size—2" to 72".

RAY BAHR

41 GREENWAY, HAMDEN, CONNECTICUT

New England Representative

Pipe Founders Sales Corporation

CAST IRON PIPE AND FITTINGS

FOR

Water, Steam, Gas and Culverts

Special Flanged and Bell and Spigot Castings

Lyncast Iron Chemical Castings

Office: 6 Beacon Street, Boston **Warehouse and Yard: East Cambridge, Mass.**

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Affiliated with Edward F. Hughes Company (Est. 1910)

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Warehouse
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1879—ROSS— 1879

AUTOMATIC VALVES

Controls elevation of water in tanks, basins and reservoirs

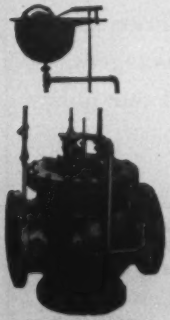
ALTITUDE VALVE

1. Single Acting
2. Double Acting

Maintains desired discharge pressure regardless of change in rate of flow

REDUCING VALVE

Regulates pressures in gravity and pump systems; between reservoirs and zones of different pressures, etc.



Maintains levels in tank, reservoir or basin

1. As direct acting

2. Pilot operated and with float traveling between two stops, for upper and lower limit of water elevation.

FLOAT VALVE

A self contained unit, with three or more automatic controls

COMBINATION VALVE

Combination automatic control both directions through the valve.

Maintains safe operating pressures for conduits, distribution and pump discharge

SURGE-RELIEF VALVE

Electric remote control—solenoid or motor can be furnished

REMOTE CONTROL VALVE

Adapted for use as primary or secondary control on any of the hydraulically controlled or operated valves.

ROSS VALVE MFG. CO., INC. Box 592 TROY, N. Y.

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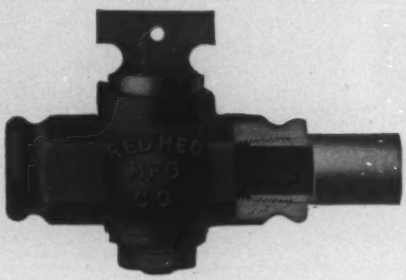

Fig. 220

It has always been the desire and determination of Red Hed Mfg. Co. to produce the best water works brass goods possible.

The continued purchase of Red Hed brass goods by a large number of water works managers, is, we believe, proof of the correctness of our policy which will be continued in future.

RED HED MFG. CO.

Makers of

RED HED Brass Goods for Water Works

368 Congress Street **Boston 10, Mass.**

Sixty-seventh

ANNUAL CONVENTION

of

THE NEW ENGLAND

WATER WORKS ASSOCIATION

September 14 - 17, 1948

Hotel Pennsylvania

New York City, N. Y.

IN JOINING BELL & SPIGOT WATER MAIN

Tegul-
MINERALEAD

CUTS

- time needed to lay the line
- delay in starting back-filling
- labor and maintenance costs

THE addition of olefine polysulphide gives super resistance to mechanical shock and abrupt temperature changes, also reduces shrinkage in cooling • Ingot is handy to work with, impervious to moisture and cannot change its correct composition. • Full information from

THE ATLAS MINERAL PRODUCTS CO. of Pa.
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Manufacturers also of
the HYDE-RO Ring and G—K, the *Original*
Bituminous Sewer Joint Compound

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WORCESTER, MASSACHUSETTS

Attention Mr. Water Works Superintendent ! !

CEMENT LINED SERVICE PIPE:—

Resists Corrosion

Has proven practical for nearly three-quarters of a century

Immediate delivery can be made to waterworks on both wrought iron and steel, black or galvanized.

"SPECIFY EUREKA PIPE"

EUREKA PIPE COMPANY, Inc.

Manufacturers of CENTRIFUGAL CEMENT LINED PIPE and LEAD AND TIN LINED FITTINGS

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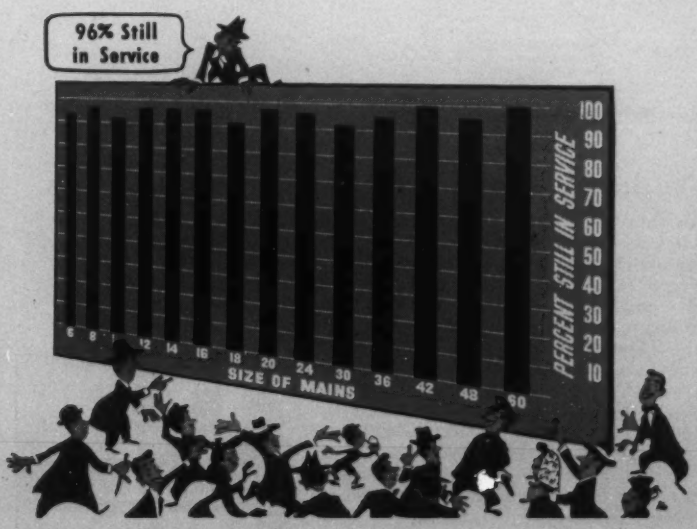

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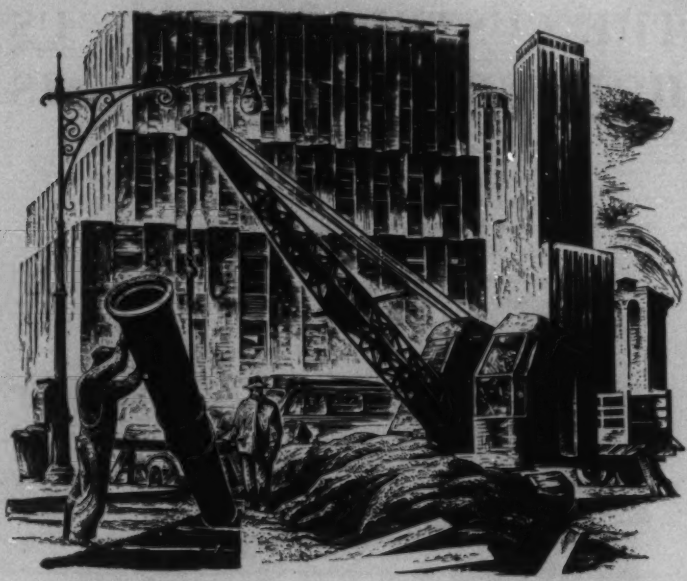

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